

Tab IV – Structural

**DAM SAFETY ASSURANCE PROGRAM
EVALUATION REPORT AND
ENVIRONMENTAL IMPACT STATEMENT**

**APPENDIX C – TAB IV
STRUCTURAL CALCULATIONS**

**DOVER DAM, OH
TUSCARAWAS RIVER**

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1 General

Structural stability of the dam was checked using two-dimensional limit equilibrium methods to check for overturning, sliding, and bearing failure modes. Much of these analysis techniques are based on those used for the Bluestone DSA Project which have evolved over the last several years.

For this report three representative monoliths were chosen for analysis. These monoliths represented the left and right non-overflow sections and the spillway section of the dam and are discussed in more detail below. These monoliths were first checked against existing conditions to determine the stability of the dam in its current state. This analysis was used to determine the Imminent Failure Flood (IFF) curve which will be used to develop Interim Risk Reduction Measures. These same models were then used to design remedial measures for the selected plan.

Other appurtenant features of the selected plan utilized typical designs and/or designs from previous similar projects. These and the stability analysis are described in more detail below.

2 References

Below is a table of documents referenced during the structural analysis of the dam as well as some other documents that will be required for the final design of the recommended plan. In addition to these documents other pertinent data can be found in the Hydrology and Hydraulic Tab and Geotechnical Tab of this appendix.

Title	Date
EC 1110-2-6061 Engineering and Design: Safety of Dams – Policy and Procedures	30 April 2004
ER 1110-2-1150 Engineering and Design for Civil Works Projects	31 August 1999
EM 1110-2-2100 Stability Analysis of Concrete Structures	1 December 2005
ERDC/ITL TR-00-1 Evaluation and Comparison of Stability Analysis and Uplift Criteria for Concrete Gravity Dams by Three Federal Agencies	January 2000
EM 1110-2-2104 Strength Design for Reinforced-Concrete Hydraulic Structures	20 August 2003
EM 1110-2-2105 Design of Hydraulic Steel Structures	31 May 1994
EM 1110-2-2200 Gravity Dam Design	30 June 1995
EM 1110-2-2502 Retaining and Flood Walls	29 September 1989
EM 1110-2-2504 Design of Sheet Pile Walls	31 March 1994
EM 1110-2-6050 Response Spectra and Seismic Analysis of Concrete Hydraulic Structures	30 June 1999
EM 1110-2-6051 Engineering and Design – Time-History Dynamic Analysis of Concrete Hydraulic Structures	22 December 2003
ER 1110-2-1806 Earthquake Design & Evaluation of Civil Works Projects	31 July 1995

EM 1110-1-2908 Rock Foundations	30 November 1994
ETL 1110-2-256 Sliding Stability for Concrete Structures	24 June 1981
Instruction Report ITL-87-5 Sliding Stability of Concrete Structures (CSLIDE)	October 1987
Bluestone DSA Project DDR's and P&S's	Various

Table IV-1. References

3 Structural Analysis

3.1 Design and Analysis Techniques and Assumptions

EM 1110-2-2100 was used as the basis for the stability analysis of Dover Dam. Various other documents listed in Table IV-1 above will be used in the analysis and design of other appurtenant features. These are discussed in more detail below.

The two-dimensional limit equilibrium analysis includes hydrostatic pressures from head water and tail water, uplift forces along the failure planes, and gravity forces. Also included for the spillway section are forces due to crest pressures during spillway flows. Hydrodynamic forces were applied to the baffles for the spillway section as well. Three-dimensional effects, particularly load transfer between monoliths, are neglected for this study and are not easily quantifiable.

The stability calculations were performed using Microsoft Excel spreadsheets with various add-on applications. The cross sectional area of the dam and its center of gravity were calculated with Bentley Microstation CADD drawings. Crest pressures for various pool elevations provided by CELRH-EC-WH were input into a structural analysis program onto a simple frame model to calculate resultant forces and moments to be applied to the stability analysis. Linear interpolation was used to develop these forces at different elevations. Hydrodynamic forces on the baffles were calculated using a formula from FEMA. Both of these hydraulic loads will be better defined through physical hydraulic modeling during the design phase of the project.

Overturning analysis was performed by summing forces and moments about the toe of the dam. Uplift forces utilized are consistent with the methods prescribed in EM 1110-2-2100 and ERDC/ITL TR-00-1 with a drain efficiency of 5% per foundation drain for the existing condition and 0% for the design of the anchors. An iterative process is built into the spreadsheet to determine the length, if any, of a tension crack on which 100% of headwater is applied. Drain efficiency is discussed in further detail in Tab II – Geotechnical.

Sliding analysis was performed using wedge methods and equations described in ITL-87-5. One exception is that at-rest pressures were used for soil loads as opposed to active or passive wedges. This is not considered to be over-conservative due to the rigidity of the structure and minimal movement prior to reaching failure and/or design limits.

Foundation pressures for bearing analysis were calculated based on a formula for a rigid structure on a rigid foundation with no tensile capacity. Bearing capacity was supplied by CELRH-EC-GG and further discussion is included in Tab II – Geotechnical.

Only global stability was analyzed for this report since the geometry of the dam and the loading conditions do not lend themselves to internal stability issues. The cross section of the dam at various elevations appears to be wide relative to the pool elevations in the anticipated load cases. This will be further investigated during the design phase of the project but this is not anticipated to require any remedial measures.

The required stability factors of safety used for the design of the recommended plan are those listed in Chapter 3 of EM 1110-2-2100. Dover Dam is defined as a critical structure with ordinary site information per this chapter. These factors of safety are as follows:

Load Case	Base in Compression	Sliding FS	Bearing Pressure Limit	Floatation FS
Usual	100%	2.0	1.0•Allow.	1.3
Unusual	75%	1.5*	1.15•Allow.	1.2
Extreme	>0%	1.1*	1.5•Allow.	1.1

* For seismic analysis FS=1.7 for unusual and FS=1.3 for extreme

Table IV-2. Required Factors of Safety

3.2 Imminent Failure Flood

The existing conditions of the dam were modeled at one foot increments of both head water and tail water to determine at what combinations the dam would reach specific factors of safety. Based on the analysis performed, the spillway monolith analyzed (Monolith 7) controlled the IFF. Figure IV-1 shows a plot of curves identifying pool combinations that yield factors of safety of 1.0 (IFF), 1.1, and 1.2. Also noted on the plot is the pool combination at the pool of record. The minimum factor of safety during this event was approximately 1.12 based on the failure mechanism and model used for this report. This analysis contains a reasonable amount of conservatism due to the number of unknowns, the critical nature of the analysis, and the severe consequences of a dam failure.

These numbers are currently being used to develop Interim Risk Reduction Measures (IRRM). Any measures developed will be coordinated with the Dam Safety Committee, Emergency Management personnel (USACE and local), and Dam Operation personnel.

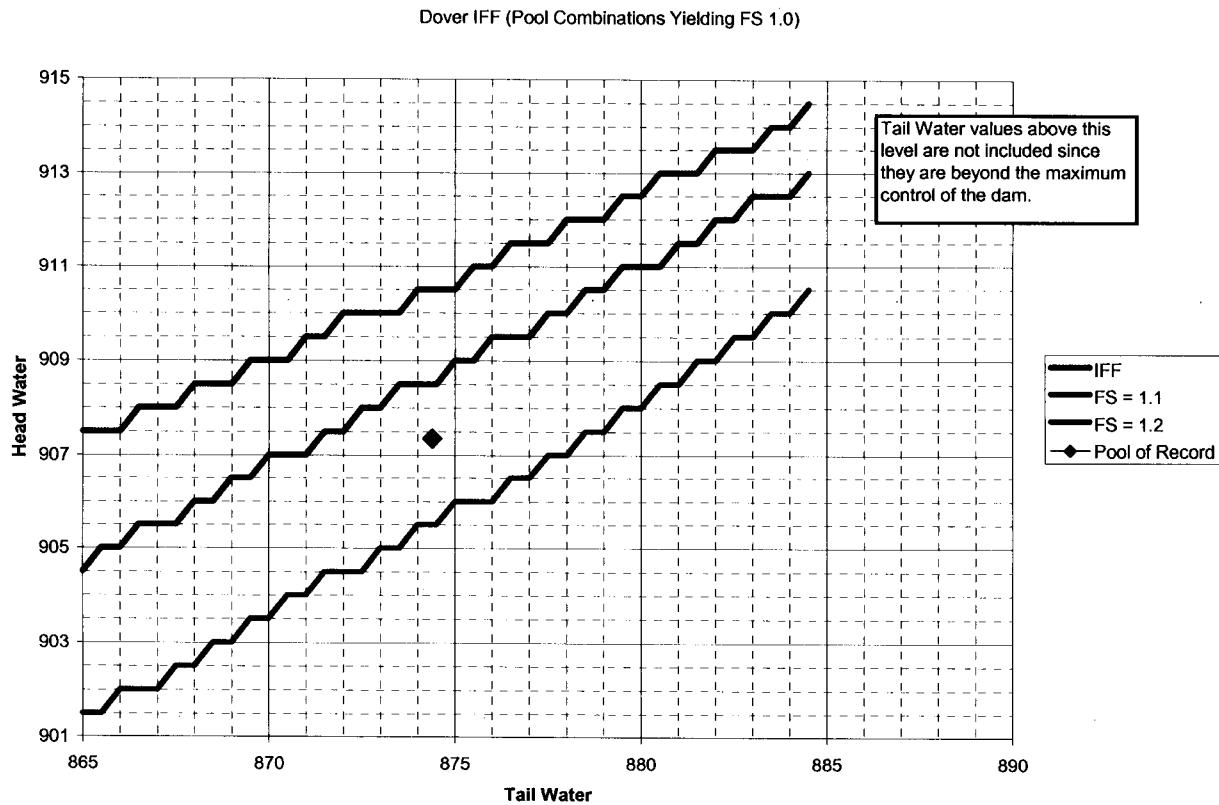


Figure IV-1. Dover Dam IFF Curves

3.3 Load Cases

Three load cases were considered for the design of the fix. The first load case is pool to the crest of the spillway (916.0) with minimum tail water (868). This would allow all of the gates to remain near closed up until the point spillway flows would occur. This pool elevation represents approximately a 185 year event therefore is considered an unusual load case. The next load case considered was a 300 year event which corresponds to the greatest return period considered an unusual load case. This was a pool elevation of 918.3 and the corresponding tail water from the spillway rating curve was 881.4. The final load case was the 100% PMF case which is the recommended design level for the structure. Pool elevation for this case is 937.7 with a corresponding tail water of 907.5. This is considered an extreme event and was the controlling load case for all three monoliths analyzed.

Additional load cases may be warranted during the design phase depending on results from physical hydraulic modeling. A normal load case and seismic load case will also be investigated but are not expected to control the design. See paragraph 4 below for more information on Seismic Studies for the dam. However, the load cases analyzed for this report should be adequate to capture the scope and quantities for the Baseline Cost Estimate.

3.4 Right Non-Overflow Section (Monolith 5)

Monolith 5 was chosen as the most critical for this portion of the dam due to its geometry. This monolith meets current criteria for all three load cases assuming the downstream embankment remains intact and does not require any stabilizing anchors. This monolith includes one foundation drain therefore 5% drain efficiency was assumed for this analysis. The remaining monoliths on this portion of the dam are assumed to meet criteria as well but will be further investigated during the design phase of the project. Results for this monolith are listed in Table IV-3.

Load Case	Head Water	Tail Water	Base in Comp.	Sliding FS	Bearing FS
1	916.0	868.0	100%	6.5	3.1
2	918.3	881.4	100%	13.1	4.3
3	937.7	907.5	39.1%	6.1	2.2

Table IV-3. Monolith 5 Results

3.5 Spillway Section (Monolith 7)

Monolith 7 was chosen as the most critical for this portion of the dam. This monolith was found to be the most critical in previous studies of the dam. Two main failure planes were investigated for this monolith. One passes through the concrete key and through the shale layer beneath the foundation and daylights downstream of the stilling basin. The second is deep seated sliding beneath the key and daylights along a fault and shears through the concrete of the stilling basin. The first failure plane proved to be the critical case.

All of the spillway monoliths include two foundation drains each therefore 10% drain efficiency was assumed for the initial analysis. However, since the stilling basin will require extensive anchoring which will essentially produce a grout curtain, 0% drain efficiency is used for the design of the monolith anchors. This is discussed in more detail in Tab II – Geotechnical.

For the analysis of the existing conditions of the dam, a passive wedge was included downstream of the stilling basin for failure plane one. However, since there is no indication that the stilling basin can handle significant spillway flows this passive wedge is assumed to erode away and no resisting force is included for the design of the anchors. An erosion cut-off wall is included in the recommended plan at the toe of the stilling basin and is discussed in more detail in paragraph 3.8. Since the extent of erosion can not be determined until physical hydraulic modeling, the cut off wall was not designed for strength. Therefore, the current design does not include any shear resistance from this wall but this will be investigated in more detail during the design phase.

Two other critical assumptions are included in the analysis of this monolith. The first is the shear strength of the concrete key. The second is the strain compatibility between the concrete key and the bedding planes of the shale layer.

For the strength of the concrete key a friction angle of 38 degrees was used and the cohesion was purely based on the shear strength of the reinforcing steel. Testing was not performed on the concrete portions of core samples taken in previous drilling programs since the core size was too

small. However, the cores indicated non-intact lift joints, cold joints and areas of poor consolidation. Not much is known about the reinforcing other than the sizes and configuration shown on available as-built drawings and the allowable stresses used in the original design computations. This original allowable stress of 14 ksi was used as the shear capacity of the reinforcing bars. Using $0.4*F_y$ would give an equivalent yield stress of 35 ksi for the reinforcing bars which is reasonable for steel of this era.

To account for the strain compatibility between the rock and concrete key, the shear strength of the rock was reduced. Shear deformation needed for the reinforcing steel to reach its peak strength is much less than that of a natural fracture in shale. At the time the steel has reached its peak strength the natural fracture of the shale will not have reached its peak strength, therefore the natural fracture strength that was derived from the plot of the peak shear strengths is not appropriate and it was reduced by 50%. The steel in the reinforced key will reach (peak strength) failure before the natural fracture of the rock because the steel is expected to take less deformation in shear to develop its peak strength than a natural fracture in shale. This is also discussed in more detail in Tab II – Geotechnical.

This monolith, in its existing condition, does not meet criteria for any of the three load cases, therefore, anchors will be required to stabilize the monolith. The geometry of the monolith restricts the location of any anchors to the centerline of the monolith. Three anchors are required to stabilize the monolith which are spaced evenly on the face of the spillway. These anchors are 59-strand, 7-wire, prestressed anchors placed at 45 degrees from vertical. The design strength of the anchors is assumed to be 60% of the ultimate tensile strength. Details of these anchors are similar to those being placed for the Bluestone DSA project and a sketch is included at the end of this Tab. Embedment depth was designed by CELRH-EC-GG and is discussed in Tab II – Geotechnical.

Each of the 9 spillway monoliths has slightly different foundation conditions and geometry. All of these will be analyzed separately during the design phase but for this report each monolith is assumed to require the same number and size of anchors to meet stability requirements. This represents a total of 27, 59-strand anchors for the spillway. These stability computations are included at the end of this Tab with the results summarized in Tables IV-4 and IV-5.

Load Case	Head Water	Tail Water	Base in Comp.	Sliding FS	Bearing FS
1	916.0	868.0	100%	0.74	11.22
2	918.3	881.4	100%	0.75	11.5
3	937.7	907.5	37%	<1.0*	6.01

*Practical solution not found with model.

Table IV-4. Monolith 7 Results – Existing Conditions

Load Case	Head Water	Tail Water	Base in Comp.	Sliding FS	Bearing FS
1	916.0	868.0	100%	1.6	6.4
2	918.3	881.4	100%	1.9	6.5
3	937.7	907.5	100%	1.1	12.2

Table IV-5. Monolith 7 Results – With Anchors

3.6 Left Non-Overflow Section (Monolith 17)

Monolith 17 was chosen as the most critical for this portion of the dam due to its geometry. None of the monoliths on the left non-overflow section of the dam contain foundation drains. Therefore, drain efficiency assumptions do not affect this portion of the dam. Even so, this monolith meets all current criteria for stability. This is assumed to hold true for monoliths 16 through 23 so no anchors are included in the selected plan. Each of these monoliths will be analyzed during the design phase to verify this assumption. Stability computations for Monolith 17 are included at the end of this Tab.

Load Case	Head Water	Tail Water	Base in Comp.	Sliding FS	Bearing FS
1	916.0	868.0	100%	5.0	3.6
2	918.3	881.4	100%	6.6	4.8
3	937.7	907.5	93.1%	6.0	3.3

Table IV-6. Monolith 17 Results

3.7 Stilling Basin

Anchors for the stilling basin were conservatively designed to meet floatation factors of safety using 60% tail water retrogression during spillway flows, as described in Paragraph 3-3 of EM 1110-2-2200, as well as 100% uplift. Anchoring of the stilling basin also allows for it to provide sliding resistance to the spillway monoliths. These anchors are assumed to be 150 ksi bar anchors as opposed to strand anchors so they may be installed in the wet. These anchors will be refined during the design phase based on findings from physical hydraulic modeling. Also, for the analysis in this report, the stilling basin was considered to act as a single unit although it was constructed in three sections (upstream to downstream). Future designs will likely consider the three separate sections as separate monolithic structures. Sliding analysis of the individual sections will also be considered.

3.8 Erosion Cut-Off Wall

As discussed above, an erosion cut-off wall is assumed to be required to prevent erosion of the stream bed downstream of the stilling basin from undermining the dam. This is assumed to be constructed with overlapping six feet diameter drilled shafts and will be located at the toe of the stilling basin.. These shafts are to be constructed in the wet by drilling and excavating alternating shafts to an elevation of 830 and filling with tremie concrete. No structural calculations were performed for the wall at this time. Quantities for the baseline cost estimate also include a reinforcing cage in every other shaft. The required depth of the shafts will be refined during the design phase through physical hydraulic modeling and possibly additional borings within the reach of the wall. Also during the design phase this wall will be designed further to provide additional sliding shear resistance for the spillway monoliths. This should allow for the reduction of the number and/or size of stabilizing anchors in the monoliths.

3.9 Parapet Wall

Simple calculations were performed to determine an approximate size and reinforcing requirement for the parapet wall. Design of this wall is controlled by EM 1110-2-2104. This

wall should be attached to the upstream face of the dam to prevent flood waters from inundating the top surface. This would allow access to this area during a flood and would eliminate the need to waterproof several access points and openings to the inspection gallery. Since the size and purpose of this wall is very similar to that used for the Bluestone DSA Project, those details, including the connection to the dam, were used to develop the Baseline Cost Estimate.

It is recognized that architectural treatment of this wall may be required to maintain the historical nature of the project and this will be addressed in the design phase. Also, the portion on the reach of the operation building would likely be designed as to not affect the upstream (outside) face of the building and should only require minimum buttressing on the inside and waterproofing of the windows.

3.10 I-Wall

The layout of the raised portion of the dam turns upstream away from the face of the dam before it reaches high ground on both abutments. It is assumed these portions will be designed and constructed as I-wall type structures based on the requirements of EM 1110-2-2502. However, consideration has been given to findings from studies of recent failures of I-Wall type structures. These potential failure modes will be investigated in the final design of the wall but are not expected to alter the final configuration of the conservative typical sections used. These typical sections, developed for similar level of design of flood protection structures, were used to estimate this portion of the selected alternative. However, it was determined that driving sheet piling on the right abutment would be impractical due to the stone fill. Therefore this reach is assumed to be founded on drilled H-piles encased in concrete spaced on 6 feet centers. These piles were sized to have equivalent bending properties as the assumed sheet piling. A toe drain on the downstream side would possibly be required to intercept seepage.

3.11 Gate Closure

Since the top of the raised portion of the dam is higher than the high point on the adjacent Route 800 a gate closure will be required at this location. This gate closure is assumed to be a typical, structural steel, swing gate closure and existing gates of similar size were used to estimate the scope and cost. The gate width will be approximately 25 feet and the height approximately 4 feet.

4 Seismic Analysis

Since Monolith 7 appears to be the critical monolith its existing condition was analyzed for both the Operational Basis Earthquake (OBE) and the Maximum Credible Earthquake (MCE). The method chosen for this analysis was the Seismic Coefficient Method as described in ER 1110-2-1806. Since Dover Dam has a low seismic hazard and this report is a feasibility level document, this is an appropriate level of analysis.

For both the OBE and MCE the seismic coefficient used was equivalent to the Peak Horizontal Acceleration (PHA). These values were 0.10g and 0.15g, respectively. The analysis assumes the coincident pool to be 870 (run of river) with minimum tailwater of 868. The seismic load is assumed to act in the downstream direction. Hydrodynamic forces and concrete voids were

neglected but should be minor and counteract each other. The monolith remained 100% base in compression for both cases and had sliding factors of safety of 2.32 and 1.77 for the OBE and MCE, respectively.

Therefore the dam appears to be safe against seismic loading. A more detailed analysis will be conducted during the design phase to ensure that an earthquake in combination with anchoring of the dam will not cause the dam to slide in the upstream direction. More information on seismic studies for Dover Dam and the determination of the OBE, MCE, and PHA are included in Tab II – Geotechnical.

5 Summary

Limited structural analysis was performed for this report. However, the most critical features were analyzed which confirmed the significant stability concerns with Dover Dam. This analysis allowed for the formulation of the recommended plan to an appropriate level for development of a base line cost estimate.

The key structural features of the recommended plan include the following:

Item	Quantity
59-Strand Prestressed Anchors (Spillway)	27 ea.
2" Dia. Prestressed Bar Anchors (Stilling Basin)	130 ea.
Parapet Wall (8 ft. Average Height)	380 Lin. Ft.
I-Wall (7 ft. Average Height)	350 Lin. Ft.
Gate Closure (25 ft. by 4 ft.)	1 ea.
Erosion Cut-Off Wall (Concrete Drilled Shafts)	91 ea.

Table IV-5. Structural Features

The following list outlines the major design and/or analysis remaining to be performed during the design phase:

- Detailed stability analysis and design of all 23 monoliths.
- Detailed stability analysis of the stilling basin as individual sections.
- Detailed design of the parapet and I-walls.
- Detailed design of the gate closure.
- Seismic analysis of the dam.
- Internal stress analysis resulting from large prestressed anchors in the dam.
- Detailed design of the erosion cut-off wall.

6 Structural Calculations

The following pages are the key structural calculations performed for this report and select as-built drawings. Preliminary calculations are not included, nor are calculations for features eliminated from consideration.

Monolith 5

Case 1: Spillway Crest

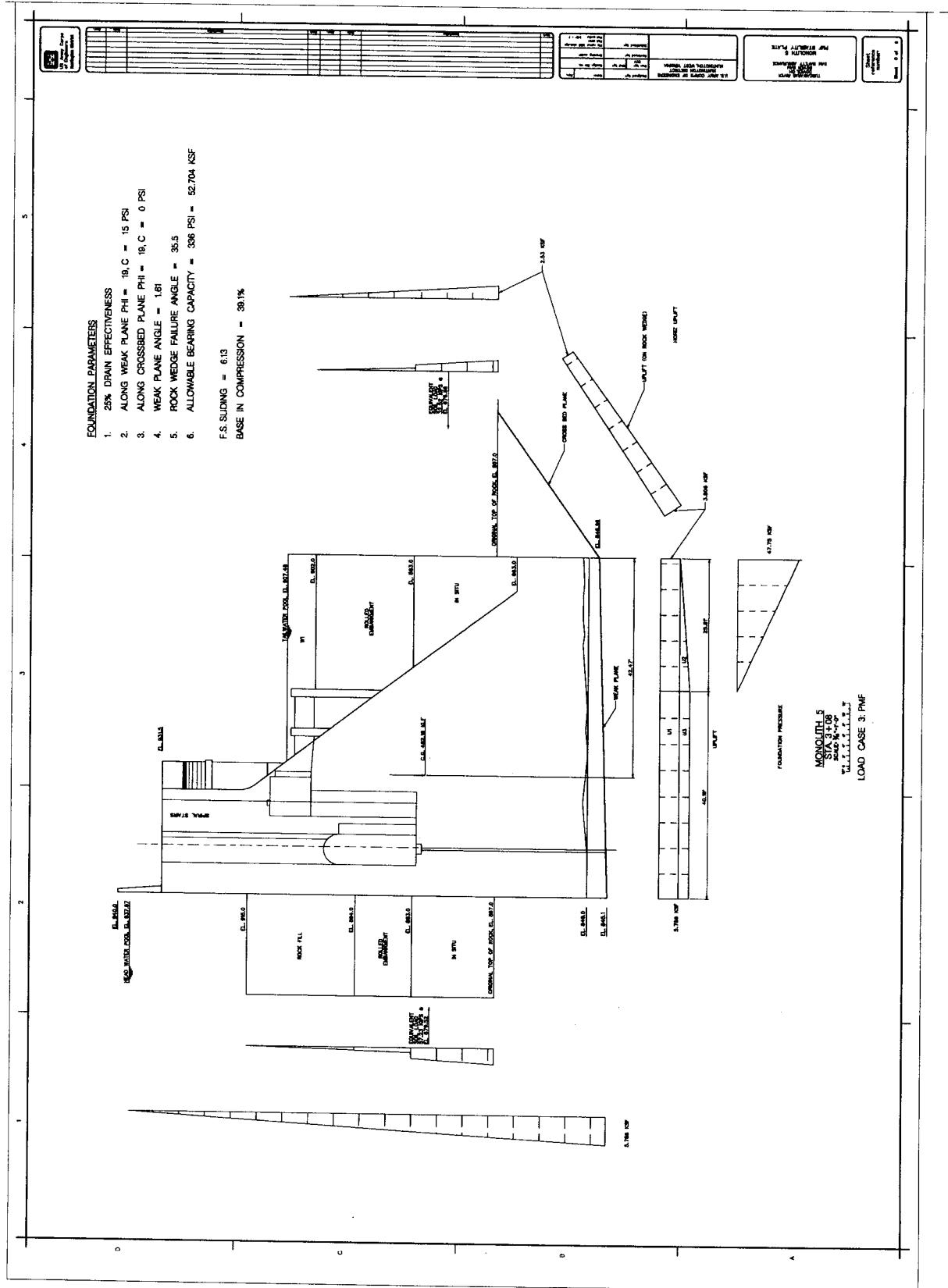
Head water	EL 916.0
Tail water	EL 868.0
Base in Compression	100 %
F.S. Sliding	6.5
F.S. Bearing	3.1

Case 2: 300 year event

Head water	EL 918.3
Tail water	EL 881.4
Base in Compression	100 %
F.S. Sliding	13.1
F.S. Bearing	4.3

Case 3: PMF

Head water	EL 937.67
Tail water	EL 907.48
Base in Compression	39.1 %
F.S. Sliding	6.1
F.S. Bearing	2.2



DOVER DSA MONOLITH 5

INPUTS		Var. Name	Notes
Head Water Elevation	916	HW	
Tail Water Elevation	868	TW	
Failure Plane Elevation	845.1	BaseEI	
Failure Plane Slope	1.61	BaseS	
Toe Elevation	846.95	ToeEL	
Drain Efficiency	5%	DrainEff	
Top Elevation	931.5	TopEl	
Base Width	66	BaseW	
Length of Monolith	36	MonLength	
Analysis Length	1	Length	
Backfill Elevation U/S	915	BFUS	
Backfill Elevation D/S	902	BFDS	
Backfill Ko		Ko	
Backfill Gamma moist		gamma_m	in kcf
Backfill Gamma Sat.		gamma_sat	in kcf
Failure Plane Cohesion	15.00	BaseC	in psi
Failure Plane phi	19.00	BasePhi	
Crossbed Cohesion	0.00	XBedC	
Crossbed Phi	19.00	XBedPhi	
Crossbed Failure Angle	35.50	XBedAng	
Gamma Rock	0.1685	gamma_r	
Top Rock US	867.00	RockUS	Must be greater than or equal to Base Elevation
Top Rock DS	867.00	RockDS	Must be greater than or equal to Base Elevation
Bearing Capacity	52.7	BearCap	
Row 1 Anchors			
# of Anchors per Mono	0		
# of Strands per Anchor	0		270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	60.0		Angle down from horizontal in degrees
Elevation of Anchors	896.0	AnchEI	Elevation Anchors are Installed
Distance from Toe	32.3	AnchLoc	Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	0.00	V_Anch	Calculated in Kips per Length
Horizontal Anchor Force	0.00	H_Anch	Calculated in Kips per Length
Row 2 Anchors			
# of Anchors per Mono	0		
# of Strands per Anchor	0		270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	60.0		Angle down from horizontal in degrees
Elevation of Anchors	849.0	AnchEI2	Elevation Anchors are Installed
Distance from Toe	0.0	AnchLoc2	Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	0.00	V_Anch2	Calculated in Kips per Length
Horizontal Anchor Force	0.00	H_Anch2	Calculated in Kips per Length
Row 3 Anchors			
# of Anchors per Mono	0		
# of Strands per Anchor	0		270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	90.0		Angle down from horizontal in degrees
Elevation of Anchors	849.0	AnchEI3	Elevation Anchors are Installed
Distance from Toe	0.0	AnchLoc3	Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	0.00	V_Anch3	Calculated in Kips per Length
Horizontal Anchor Force	0.00	H_Anch3	Calculated in Kips per Length

GAM SAT GAM M PHI

ROCK FILL	0.130	0.120	35
ROLLED EMBANKMEN	0.125	0.120	33
IN SITU	0.128	0.125	0



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Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 5 - Right Abutment
Deducts*

Computed By: SAW Date 28 Nov 2006

Checked By: ECV Date 28 Nov 2006

Item	Width	Height	Length	Volume	Arm	Moment
Gallery - Rect. Portion	6.00 X	15.00 X	36.00 X	3240.00 X	57.00 =	184680.0
Gallery - Arch Portion	6.00 X	3.00 X	36.00 X	508.94 X	57.00 =	29009.5
Bulkhead Storage	2.00 X	15.00 X	6.67 X	200.00 X	53.00 =	10600.0
Spiral Stairway	6.17 X	49.25 X	6.17 X	1470.95 X	50.92 =	74895.7
Stairs to D/S Access	5.00 X	8.00 X	18.00 X	720.00 X	48.19 =	34696.8
D/S Access - Rect. Portion	7.90 X	8.00 X	8.00 X	505.60 X	46.74 =	23631.7
D/S Access - Tri. Portion	8.00 X	8.00 X	8.00 X	256.00 X	40.12 =	10271.6
	X	X	X	0.00 X	=	0.0
				6901.49		367785.3
Equivalent Square Deduct	13.85 X	13.85 X	36 X	6901.49 X	53.29 =	367785.3



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COMPUTATION

*Monolith 5 - Right Abutment
Load Case 1 at Failure Plane Elevation 845.1 to 846.9 ft*

							Computed By:	ECV	Date	28 Nov 2006
							Checked By:	SAW	Date	28 Nov 2006
Width	X	Height	X	Unit Wt.	X	Length	Vertical	Horizontal	Arm	Moments
C1										
CONCRETE										
C1	Dam Concrete from Microstation						482.19			20,477.14
C2	13.85 X 13.85 X 0.145 X 1 X -1						(27.80)			(1,481.36)
	Concrete Subtotal =						454.39			18,995.78
Anchor Forces										
AV1	0 - 0 Strand Anchors @ 60 degrees						0.00			0.00
AH1	0 - 0 Strand Anchors @ 60 degrees							0.00		0.00
AV2	0 - 0 Strand Anchors @ 60 degrees						0.00			0.00
AH2	0 - 0 Strand Anchors @ 60 degrees							0.00		0.00
AV3	0 - 0 Strand Anchors @ 90 degrees						0.00			0.00
AH3	0 - 0 Strand Anchors @ 90 degrees							0.00		0.00
MISC. VERTICAL										
S1	550.66 X 1 X 0.120 X 1 X 1						66.08			977.31
S2	241.88 X 1 X 0.125 X 1 X 1						30.23			253.67
S3	43.125 X 1 X 0.128 X 1 X 1						5.52			24.19
S4	X X X 1 X 1							0.000		
R1	66 X 3.9 X 0.169 X 1 X 1						43.37			1,431.27
UPLIFT										
U1	66 X 21.05 X 0.0625 X 1 X -1						(86.83)			
U2	57 X 40.9 X 0.0625 X 1 X -0.5						(72.85)			
U3	0 X 49.85 X 0.0625 X 1 X -1						0.00			0.00
U4	9 X 8.9503 X 0.0625 X 1 X -0.5						(2.52)			(154.81)
U5	9 X 40.9 X 0.0625 X 1 X -1						(23.01)			(1,449.38)
U6	1.8505 X 21.05 X 0.0625 X 1 X -1							(2.43)	0.925	(2.25)
U7	1.5981 X 40.9 X 0.0625 X 1 X -0.5							(2.04)	1.065	(2.18)
U8	0 X 49.85 X 0.0625 X 1 X -1							0.00	1.850	0.00
U9	0.2523 X 8.9503 X 0.0625 X 1 X -0.5							(0.07)	1.724	(0.12)
U10	0.2523 X 40.9 X 0.0625 X 1 X -1							(0.65)	1.766	(1.14)
HYDROSTATIC										
H1	70.9 X 70.9 X 0.0625 X 1 X -0.5							(157.09)	25.483	
H2	21.1 X 21.1 X 0.0625 X 1 X 0.5							13.85	7.017	97.16
MISC. HORIZONTAL										
E1	57.233 X 1 X 1.000 X 1 X -1							(57.23)	32.570	
E2	32.521 X 1 X 1.000 X 1 X 1							32.52	30.160	
E3	X X X 1 X 1								0.000	
E4	X X X 1 X 1								0.000	
Sum V <u>414.39</u> Sum H <u>(173.15)</u> Sum M <u>9,649.30</u>										
$M/V = 23.29 \text{ ft.}$ $e = M/V-B/2 = -9.71 \text{ ft.}$ %Base in Compression = 100.0%										
Max. Found. Pressure= 17.108 ksf Bearing Capacity= 52.704 ksf										
Sliding F.S.= 6.459 Bearing F.S.= 3.081										

Wedge Analysis

FS =	6.459
	0.950
Inputs	
Crossbed c =	0.000
Crossbed ϕ =	19.000
rad =	0.332
Unit Weight of Rock =	0.169
c at Base =	15.000
ϕ at Base =	19.000
rad =	0.332
Top Rock U/S	867.00
Top Rock D/S	867.00
Upper Pool	916.00
Lower Pool	868.00

Active Wedge	
αa =	-46.526
rad =	-0.812
Wa =	
Va =	
Ua =	
La =	
Pa =	0.000
Sum =	0.000

Passive Wedge	
αp =	35.500
rad =	0.620
Wp =	18.771
Vp =	87.882
Up =	-16.681
Lp =	24.628
Hi-Hr =	-9.687
Pp =	95.814

Structure Wedge	
αs =	1.606
rad =	0.028
Ws =	454.392
Vs =	101.834
Us =	-185.280
Ls =	66.026
Hi-Hr =	173.145
Ps =	-95.814

Analysis Results

PROGRAM CSOILP - CALCULATION OF AT-REST PRESSURES BY COMBINATION
OF "GRAVITY-TURN-ON" AND THEORY OF ELASTICITY COMPONENTS
DATE: 18-AUGUST-06 TIME: 9:36:49

* RESULTS *

I.--HEADING
'M5 DS load case 1

II.--EQUIVALENT SURCHARGE LOAD DUE TO IRREGULAR SURFACE ABOVE 902.0 (FT)
NONE

III.--PRESSURES ON WALL BELOW EL. 902.0 (FT)

ELEVATION (FT)	GRAVITY- TURN-ON PRESSURE (PSF)	ELASTICITY COMPONENT PRESSURE (PSF)	COMBINED GTO & ELASTICITY PRESSURE (PSF)
902.00	0.00	0.00	0.00
901.00	28.46	0.00	28.46
900.00	56.92	0.00	56.92
899.00	85.38	0.00	85.38
898.00	113.84	0.00	113.84
897.00	142.30	0.00	142.30
896.00	170.76	0.00	170.76
895.00	199.22	0.00	199.22
894.00	227.68	0.00	227.68
893.00	256.14	0.00	256.14
892.00	284.60	0.00	284.60
891.00	313.06	0.00	313.06
890.00	341.52	0.00	341.52
889.00	369.98	0.00	369.98
888.00	398.44	0.00	398.44
887.00	426.90	0.00	426.90
886.00	455.36	0.00	455.36
885.00	483.82	0.00	483.82
884.00	512.28	0.00	512.28
883.00+	540.74	0.00	540.74
883.00-	1187.50	0.00	1187.50
882.00	1253.00	0.00	1253.00
881.00	1318.50	0.00	1318.50
880.00	1384.00	0.00	1384.00
879.00	1449.50	0.00	1449.50
878.00	1515.00	0.00	1515.00
877.00	1580.50	0.00	1580.50
876.00	1646.00	0.00	1646.00
875.00	1711.50	0.00	1711.50
874.00	1777.00	0.00	1777.00
873.00	1842.50	0.00	1842.50
872.00	1908.00	0.00	1908.00
871.00	1973.50	0.00	1973.50
870.00	2039.00	0.00	2039.00
869.00	2104.50	0.00	2104.50
868.00	2170.00	0.00	2170.00
867.00+	2235.50	0.00	2235.50

Resultant Force for Combined Pressures = 32521.04 lbs
Resultant Location at elev. = 876.58 feet

Input Data

PROGRAM CSOILP - CALCULATION OF AT-REST PRESSURES BY COMBINATION
OF "GRAVITY-TURN-ON" AND THEORY OF ELASTICITY COMPONENTS
DATE: 18-AUGUST-06 TIME: 9:36:55

* INPUT DATA *

I.--HEADING
'M5 DS load case 1

II.--SURFACE POINTS

DIST. FROM WALL (FT)	ELEVATION (FT)
0.0	902.0

III.--SOIL LAYER DATA

<WEIGHT SAT.	(PCF)> MST.	INTERNAL FRICTION (DEG)	ELASTICITY COEFF.	BOTTOM ELEV. (FT)
125.0	120.0	33.0	0.00	883.0
128.0	125.0	0.0	0.00	867.0

IV.--WATER DATA

UNIT WEIGHT:	62.5 (PCF)
ELEVATION:	916.0 (FT)

DOVER DSA MONOLITH 5

INPUTS	Var. Name	Notes
Head Water Elevation	918.3	HW
Tail Water Elevation	881.4	TW
Failure Plane Elevation	845.1	BaseEI
Failure Plane Slope	1.61	BaseS
Toe Elevation	846.95	ToeEL
Drain Efficiency	5%	DrainEff
Top Elevation	931.5	TopEl
Base Width	66	BaseW
Length of Monolith	36	MonLength
Analysis Length	1	Length
Backfill Elevation U/S	915	BFUS
Backfill Elevation D/S	902	BFDS
Backfill Ko	Ko	
Backfill Gamma moist	gamma_m	in kcf
Backfill Gamma Sat.	gamma_sat	in kcf
Failure Plane Cohesior	15.00	BaseC
Failure Plane phi	19.00	BasePhi
Crossbed Cohesion	0.00	XBedC
Crossbed Phi	19.00	XBedPhi
Crossbed Failure Angle	35.50	XBedAng
Gamma Rock	0.1685	gamma_r
Top Rock US	867.00	RockUS
Top Rock DS	867.00	RockDS
Bearing Capacity	52.7	BearCap
Row 1 Anchors		
# of Anchors per Mono	0	
# of Strands per Anchor	0	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	60.0	Angle down from horizontal in degrees
Elevation of Anchors	896.0	AnchEI
Distance from Toe	32.3	AnchLoc
Vertical Anchor Force	0.00	V_Anch
Horizontal Anchor Force	0.00	H_Anch
Row 2 Anchors		
# of Anchors per Mono	0	
# of Strands per Anchor	0	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	60.0	Angle down from horizontal in degrees
Elevation of Anchors	849.0	AnchEI2
Distance from Toe	0.0	AnchLoc2
Vertical Anchor Force	0.00	V_Anch2
Horizontal Anchor Force	0.00	H_Anch2
Row 3 Anchors		
# of Anchors per Mono	0	
# of Strands per Anchor	0	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	90.0	Angle down from horizontal in degrees
Elevation of Anchors	849.0	AnchEI3
Distance from Toe	0.0	AnchLoc3
Vertical Anchor Force	0.00	V_Anch3
Horizontal Anchor Force	0.00	H_Anch3

	GAM SAT	GAM M	PHI
ROCK FILL	0.130	0.120	35
ROLLED EMBANKMENT	0.125	0.120	33
IN SITU	0.128	0.125	0



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 5 - Right Abutment
Deducts*

Computed By: SAW Date 28 Nov 2006

Checked By: ECV Date 28 Nov 2006

Item	Width	Height	Length	Volume	Arm	Moment
Gallery - Rect. Portion	6.00 X	15.00 X	36.00 X	3240.00 X	57.00 =	184680.0
Gallery - Arch Portion	6.00 X	3.00 X	36.00 X	508.94 X	57.00 =	29009.5
Bulkhead Storage	2.00 X	15.00 X	6.67 X	200.00 X	53.00 =	10600.0
Spiral Stairway	6.17 X	49.25 X	6.17 X	1470.95 X	50.92 =	74895.7
Stairs to D/S Access	5.00 X	8.00 X	18.00 X	720.00 X	48.19 =	34696.8
D/S Access - Rect. Portion	7.90 X	8.00 X	8.00 X	505.60 X	46.74 =	23631.7
D/S Access - Tri. Portion	8.00 X	8.00 X	8.00 X	256.00 X	40.12 =	10271.6
	X	X	X	0.00 X	=	0.0
				<hr/> 6901.49		<hr/> 367785.3
Equivalent Square Deduct	13.85 X	13.85 X	36 X	6901.49 X	53.29 =	367785.3



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 5 - Right Abutment
Load Case 2 at Failure Plane Elevation 845.1 to 846.95*

							Computed By:	ECV	Date	28 Nov 2006
							Checked By:	SAW	Date	28 Nov 2006
Width	X	Height	X	Unit Wt.	X	Length	Vertical	Horizontal	Arm	Moments
C1										
CONCRETE										
C1	Dam Concrete from Microstation						482.19			20,477.14
C2	13.85 X 13.85 X 0.145 X 1 X -1						(27.80)			(1,481.36)
	Concrete Subtotal =						454.39			18,995.78
Anchor Forces										
AV1	0 - 0 Strand Anchors @ 60 degrees						0.00			0.00
AH1	0 - 0 Strand Anchors @ 60 degrees							0.00		0.00
AV2	0 - 0 Strand Anchors @ 60 degrees						0.00			0.00
AH2	0 - 0 Strand Anchors @ 60 degrees							0.00		0.00
AV3	0 - 0 Strand Anchors @ 90 degrees						0.00			0.00
AH3	0 - 0 Strand Anchors @ 90 degrees							0.00		0.00
MISC. VERTICAL										
S1	550.66 X 1 X 0.120 X 1 X 1						66.08			977.31
S2	32.64 X 1 X 0.125 X 1 X 1						4.08			42.64
S3	251.16 X 1 X 0.128 X 1 X 1						32.15			238.22
S4	X X X 1 X 1									
R1	66 X 3.9 X 0.169 X 1 X 1						43.37			1,431.27
UPLIFT										
U1	66 X 34.45 X 0.0625 X 1 X -1						(142.11)			
U2	57 X 31.793 X 0.0625 X 1 X -0.5						(56.63)			
U3	0 X 38.75 X 0.0625 X 1 X -1						0.00			
U4	9 X 6.9574 X 0.0625 X 1 X -0.5						(1.96)			
U5	9 X 31.793 X 0.0625 X 1 X -1						(17.88)			
U6	1.8505 X 34.45 X 0.0625 X 1 X -1							(3.98)		
U7	1.5981 X 31.793 X 0.0625 X 1 X -0.5							(1.59)		
U8	0 X 38.75 X 0.0625 X 1 X -1							0.00		
U9	0.2523 X 6.9574 X 0.0625 X 1 X -0.5							(0.05)		
U10	0.2523 X 31.793 X 0.0625 X 1 X -1							(0.50)		
HYDROSTATIC										
H1	73.2 X 73.2 X 0.0625 X 1 X -0.5							(167.45)		
H2	34.4 X 34.4 X 0.0625 X 1 X 0.5							37.09		
MISC. HORIZONTAL										
E1	57.233 X 1 X 1.000 X 1 X -1							(57.23)		
E2	56.174 X 1 X 1.000 X 1 X 1							56.17		
E3	X X X 1 X 1									
E4	X X X 1 X 1									
Sum V <u>381.49</u> Sum H <u>(137.54)</u>										
Sum M <u>9,450.99</u>										
M/V = 24.77 ft. e = M/V-B/2 = -8.23 ft. %Base in Compression = 100.0%										
Max. Found. Pressure= 12.256 ksf Bearing Capacity= 52.704 ksf										
Sliding F.S.= 13.097 Bearing F.S.= 4.300										

Wedge Analysis

FS =	13.097		
		1.000	
Inputs			
Crossbed c =	0.000		
Crossbed ϕ =	19.000		
rad =	0.332		
Unit Weight of Rock =	0.169		
c at Base =	15.000		
ϕ at Base =	19.000		
rad =	0.332		
Top Rock U/S	867.00		
Top Rock D/S	867.00		
Upper Pool	918.30		
Lower Pool	881.40		

Active Wedge	
α_a =	-45.753
rad =	-0.799
Wa =	
Va =	
Ua =	
La =	
Pa =	0.000
Sum =	0.000

Passive Wedge	
α_p =	35.500
rad =	0.620
Wp =	18.771
Vp =	87.882
Up =	-16.681
Lp =	24.628
Hl-Hr =	-9.687
Pp =	90.622

Structure Wedge	
α_s =	1.606
rad =	0.028
Ws =	454.392
Vs =	102.308
Us =	-218.663
Ls =	66.026
Hl-Hr =	137.545
Ps =	-90.622

Analysis Results

PROGRAM CSOILP - CALCULATION OF AT-REST PRESSURES BY COMBINATION
OF "GRAVITY-TURN-ON" AND THEORY OF ELASTICITY COMPONENTS
DATE: 17-AUGUST-06 TIME: 14:46:23

* RESULTS *

I.--HEADING

'M5 DS 300 year event

II.--EQUIVALENT SURCHARGE LOAD DUE TO IRREGULAR SURFACE ABOVE 902.0 (FT)
NONE

III.--PRESSURES ON WALL BELOW EL. 902.0 (FT)

ELEVATION (FT)	GRAVITY- TURN-ON PRESSURE (PSF)	ELASTICITY COMPONENT PRESSURE (PSF)	COMBINED GTO & ELASTICITY PRESSURE (PSF)
902.00	0.00	0.00	0.00
901.00	54.64	0.00	54.64
900.00	109.29	0.00	109.29
899.00	163.93	0.00	163.93
898.00	218.57	0.00	218.57
897.00	273.22	0.00	273.22
896.00	327.86	0.00	327.86
895.00	382.50	0.00	382.50
894.00	437.15	0.00	437.15
893.00	491.79	0.00	491.79
892.00	546.43	0.00	546.43
891.00	601.08	0.00	601.08
890.00	655.72	0.00	655.72
889.00	710.36	0.00	710.36
888.00	765.01	0.00	765.01
887.00	819.65	0.00	819.65
886.00	874.29	0.00	874.29
885.00	928.94	0.00	928.94
884.00	983.58	0.00	983.58
883.00+	1038.22	0.00	1038.22
883.00-	2280.00	0.00	2280.00
882.00	2405.00	0.00	2405.00
881.40	2480.00	0.00	2480.00
881.00	2506.20	0.00	2506.20
880.00	2571.70	0.00	2571.70
879.00	2637.20	0.00	2637.20
878.00	2702.70	0.00	2702.70
877.00	2768.20	0.00	2768.20
876.00	2833.70	0.00	2833.70
875.00	2899.20	0.00	2899.20
874.00	2964.70	0.00	2964.70
873.00	3030.20	0.00	3030.20
872.00	3095.70	0.00	3095.70
871.00	3161.20	0.00	3161.20
870.00	3226.70	0.00	3226.70
869.00	3292.20	0.00	3292.20
868.00	3357.70	0.00	3357.70
867.00+	3423.20	0.00	3423.20

Resultant Force for Combined Pressures = 56174.16 lbs
Resultant Location at elev. = 877.11 feet

Input Data

PROGRAM CSOILP - CALCULATION OF AT-REST PRESSURES BY COMBINATION
OF "GRAVITY-TURN-ON" AND THEORY OF ELASTICITY COMPONENTS
DATE: 17-AUGUST-06 TIME: 14:46:28

* INPUT DATA *

I.--HEADING

'M5 DS 300 year event

II.--SURFACE POINTS

DIST. FROM WALL (FT)	ELEVATION (FT)
0.0	902.0

III.--SOIL LAYER DATA

<WEIGHT SAT.	(PCF) > MST.	INTERNAL FRICTION (DEG)	ELASTICITY COEFF.	BOTTOM ELEV. (FT)
125.0	120.0	33.0	0.00	883.0
128.0	125.0	0.0	0.00	867.0

IV.--WATER DATA

UNIT WEIGHT: 62.5 (PCF)
ELEVATION: 881.4 (FT)

DOVER DSA MONOLITH 5

INPUTS	Var. Name	Notes
Head Water Elevation	937.67	HW
Tail Water Elevation	907.48	TW
Failure Plane Elevation	845.1	BaseEl
Failure Plane Slope	1.61	BaseS
Toe Elevation	846.95	ToeEL
Drain Efficiency	5%	DrainEff
Top Elevation	931.5	TopEl
Base Width	66	BaseW
Length of Monolith	36	MonLength
Analysis Length	1	Length
Backfill Elevation U/S	915	BFUS
Backfill Elevation D/S	902	BFDS
Backfill Ko	Ko	
Backfill Gamma moist	gamma_m	in kcf
Backfill Gamma Sat.	gamma_sat	in kcf
Failure Plane Cohesior	15.00	BaseC
Failure Plane phi	19.00	BasePhi
Crossbed Cohesion	0.00	XBedC
Crossbed Phi	19.00	XBedPhi
Crossbed Failure Angle	35.50	XBedAng
Gamma Rock	0.1685	gamma_r
Top Rock US	867.00	RockUS
Top Rock DS	867.00	RockDS
Bearing Capacity	52.7	BearCap
Row 1 Anchors		
# of Anchors per Mono	0	
# of Strands per Anchor	0	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	60.0	Angle down from horizontal in degrees
Elevation of Anchors	896.0	AnchEl
Distance from Toe	32.3	AnchLoc
Vertical Anchor Force	0.00	V_Anch
Horizontal Anchor Force	0.00	H_Anch
Row 2 Anchors		
# of Anchors per Mono	0	
# of Strands per Anchor	0	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	60.0	Angle down from horizontal in degrees
Elevation of Anchors	849.0	AnchEl2
Distance from Toe	0.0	AnchLoc2
Vertical Anchor Force	0.00	V_Anch2
Horizontal Anchor Force	0.00	H_Anch2
Row 3 Anchors		
# of Anchors per Mono	0	
# of Strands per Anchor	0	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	90.0	Angle down from horizontal in degrees
Elevation of Anchors	849.0	AnchEl3
Distance from Toe	0.0	AnchLoc3
Vertical Anchor Force	0.00	V_Anch3
Horizontal Anchor Force	0.00	H_Anch3

	GAM SAT	GAM M	PHI
ROCK FILL	0.130	0.120	35
ROLLED EMBANKMENT	0.125	0.120	33
IN SITU	0.128	0.125	0



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 5 - Right Abutment
Deducts*

Computed By: SAW Date 28 Nov 2006

Checked By: ECV Date 28 Nov 2006

Item	Width	Height	Length	Volume	Arm	Moment
Gallery - Rect. Portion	6.00 X	15.00 X	36.00 X	3240.00 X	57.00 =	184680.0
Gallery - Arch Portion	6.00 X	3.00 X	36.00 X	508.94 X	57.00 =	29009.5
Bulkhead Storage	2.00 X	15.00 X	6.67 X	200.00 X	53.00 =	10600.0
Spiral Stairway	6.17 X	49.25 X	6.17 X	1470.95 X	50.92 =	74895.7
Stairs to D/S Access	5.00 X	8.00 X	18.00 X	720.00 X	48.19 =	34696.8
D/S Access - Rect. Portion	7.90 X	8.00 X	8.00 X	505.60 X	46.74 =	23631.7
D/S Access - Tri. Portion	8.00 X	8.00 X	8.00 X	256.00 X	40.12 =	10271.6
		X	X	0.00 X	=	0.0
				<hr/> 6901.49		<hr/> 367785.3
Equivalent Square Deduct	13.85 X	13.85 X	36 X	6901.49 X	53.29 =	367785.3



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

Monolith 5 - Right Abutment

Case 3: PMF Analysis at Failure Plane Elevation 845.1 to 846.9t

Computed By:	ECV	Date	28 Nov 2006
Checked By:	SAW	Date	28 Nov 2006

	Width	X	Height	X	Unit Wt.	X	Length	Vertical	Horizontal	Arm	Moments
CONCRETE											
C1	13.85 X	13.85 X	0.145 X	1 X	-1			482.19		42.467	20,477.14
C2								(27.80)		53.291	(1,481.36)
Concrete Subtotal =											
Anchor Forces											
AV1	0 - 0 Strand Anchors @ 60 degrees							0.00		32.250	0.00
AH1	0 - 0 Strand Anchors @ 60 degrees								0.00	49.050	0.00
AV2	0 - 0 Strand Anchors @ 60 degrees							0.00		0.000	0.00
AH2	0 - 0 Strand Anchors @ 60 degrees								0.00	2.050	0.00
AV3	0 - 0 Strand Anchors @ 90 degrees							0.00		0.000	0.00
AH3	0 - 0 Strand Anchors @ 90 degrees								0.00	2.050	0.00
MISC. VERTICAL											
S1	550.41 X	1 X	0.125 X	1 X	1			68.80		14.787	1,017.33
S2	285 X	1 X	0.128 X	1 X	1			36.48		7.784	283.97
W1	206.76 X	1 X	0.063 X	1 X	1			12.92		18.940	244.75
W2	X	X	X	1 X	1					0.000	
R1	66 X	3.9 X	0.169 X	1 X	1			43.37		33.000	1,431.27
UPLIFT											
U1	66 X	60.53 X	0.0625 X	1 X	-1			(249.69)		33.000	(8,239.65)
U2	25.8 X	32.04 X	0.0625 X	1 X	-0.5			(25.83)		17.200	(444.31)
U3	40.2 X	32.04 X	0.0625 X	1 X	-1			(80.50)		45.900	(3,694.98)
U4	0 X	0 X	0.0625 X	1 X	-0.5			0.00		25.800	0.00
U5	0 X	32.04 X	0.0625 X	1 X	-1			0.00		25.800	0.00
U6	1.8505 X	60.53 X	0.0625 X	1 X	-1				(7.00)	0.925	(6.48)
U7	0.7234 X	32.04 X	0.0625 X	1 X	-0.5				(0.72)	0.482	(0.35)
U8	1.1271 X	32.04 X	0.0625 X	1 X	-1				(2.26)	1.287	(2.90)
U9	0 X	0 X	0.0625 X	1 X	-0.5				0.00	0.723	0.00
U10	0 X	32.04 X	0.0625 X	1 X	-1				0.00	0.723	0.00
HYDROSTATIC											
H1	92.57 X	92.57 X	0.0625 X	1 X	-0.5				(267.79)	32.707	(8,758.44)
H2	60.5 X	60.5 X	0.0625 X	1 X	0.5				114.50	20.177	2,310.15
MISC. HORIZONTAL											
E1	57.233 X	1 X	1.000 X	1 X	-1				(57.23)	32.570	(1,864.07)
E2	32.521 X	1 X	1.000 X	1 X	1				32.52	29.630	963.60
E3	X	X	X	1 X	1					0.000	
E4	X	X	X	1 X	1					0.000	
Sum V								<u>259.95</u>	Sum H	<u>(187.99)</u>	Sum M
										<u>2,235.67</u>	

M/V = 8.60 ft.
 e = M/V-B/2 = -24.40 ft.
 %Base in Compression = 39.1%

Sliding F.S.= 6.133
 Bearing F.S.= 2.202

Max. Found. Pressure= 23.934 ksf
 Bearing Capacity= 52.704 ksf

Wedge Analysis

FS =	6.133	Active Wedge	Passive Wedge	Structure Wedge
	0.950	$\alpha_a = -46.607$	$\alpha_p = 35.500$	$\alpha_s = 1.606$
Inputs				
Crossbed c =	0.000	$rad = -0.813$	$rad = 0.620$	$rad = 0.028$
Crossbed ϕ =	19.000	$Wa =$	$Wp = 18.771$	$Ws = 454.392$
$rad =$	0.332	$Va =$	$Vp = 87.882$	$Vs = 118.204$
Unit Weight of Rock =	0.169	$Ua =$	$Up = -16.681$	$Us = -356.159$
c at Base =	15.000	$La =$	$Lp = 24.628$	$Ls = 66.026$
ϕ at Base =	19.000		$Hi-Hr = -9.687$	$Hi-Hr = 187.985$
$rad =$	0.332	$Pa = 0.000$	$Pp = 96.371$	$Ps = -96.371$
Top Rock U/S	867.00			
Top Rock D/S	867.00			
Upper Pool	937.67			
Lower Pool	907.48			
Sum = 0.000				

Input Data

PROGRAM CSOILP - CALCULATION OF AT-REST PRESSURES BY COMBINATION
OF "GRAVITY-TURN-ON" AND THEORY OF ELASTICITY COMPONENTS
DATE: 10-AUGUST-06 TIME: 14:57:11

* INPUT DATA *

I.--HEADING
'DOVER DSA
'M5 PMF - US

II.--SURFACE POINTS

DIST. FROM WALL (FT)	ELEVATION (FT)
0.0	915.0

III.--SOIL LAYER DATA

<WEIGHT SAT.	(PCF) > MST.	INTERNAL FRICTION (DEG)	ELASTICITY COEFF.	BOTTOM ELEV. (FT)
130.0	120.0	35.0	0.00	894.0
125.0	120.0	33.0	0.00	883.0
128.0	125.0	0.0	0.00	867.0

IV.--WATER DATA

UNIT WEIGHT: 62.5 (PCF)
ELEVATION: 937.7 (FT)

Analysis Results

PROGRAM CSOILP - CALCULATION OF AT-REST PRESSURES BY COMBINATION
OF "GRAVITY-TURN-ON" AND THEORY OF ELASTICITY COMPONENTS
DATE: 4-AUGUST-06 TIME: 17:16:55

* RESULTS *

I.--HEADING
'DOVER DSA
'M5 PMF - US

II.--EQUIVALENT SURCHARGE LOAD DUE TO IRREGULAR SURFACE ABOVE 915.0 (FT)
NONE

III.--PRESSURES ON WALL BELOW EL. 915.0 (FT)

ELEVATION (FT)	GRAVITY- TURN-ON PRESSURE (PSF)	ELASTICITY COMPONENT PRESSURE (PSF)	COMBINED GTO & ELASTICITY PRESSURE (PSF)
915.00	0.00	0.00	0.00
914.00	28.78	0.00	28.78
913.00	57.57	0.00	57.57
912.00	86.35	0.00	86.35
911.00	115.13	0.00	115.13
910.00	143.92	0.00	143.92
909.00	172.70	0.00	172.70
908.00	201.49	0.00	201.49
907.00	230.27	0.00	230.27
906.00	259.05	0.00	259.05
905.00	287.84	0.00	287.84
904.00	316.62	0.00	316.62
903.00	345.40	0.00	345.40
902.00	374.19	0.00	374.19
901.00	402.97	0.00	402.97
900.00	431.75	0.00	431.75
899.00	460.54	0.00	460.54
898.00	489.32	0.00	489.32
897.00	518.10	0.00	518.10
896.00	546.89	0.00	546.89
895.00	575.67	0.00	575.67
894.00+	604.46	0.00	604.46
894.00-	645.47	0.00	645.47
893.00	673.93	0.00	673.93
892.00	702.39	0.00	702.39
891.00	730.85	0.00	730.85
890.00	759.31	0.00	759.31
889.00	787.77	0.00	787.77
888.00	816.23	0.00	816.23
887.00	844.69	0.00	844.69
886.00	873.15	0.00	873.15
885.00	901.61	0.00	901.61
884.00	930.07	0.00	930.07
883.00+	958.53	0.00	958.53
883.00-	2105.00	0.00	2105.00
882.00	2170.50	0.00	2170.50
881.00	2236.00	0.00	2236.00
880.00	2301.50	0.00	2301.50
879.00	2367.00	0.00	2367.00
878.00	2432.50	0.00	2432.50
877.00	2498.00	0.00	2498.00
876.00	2563.50	0.00	2563.50

Analysis Results

875.00	2629.00	0.00	2629.00
874.00	2694.50	0.00	2694.50
873.00	2760.00	0.00	2760.00
872.00	2825.50	0.00	2825.50
871.00	2891.00	0.00	2891.00
870.00	2956.50	0.00	2956.50
869.00	3022.00	0.00	3022.00
868.00	3087.50	0.00	3087.50
867.00+	3153.00	0.00	3153.00

Resultant Force for Combined Pressures = 57232.83 lbs
Resultant Location at elev. = 879.52 feet

Input Data

PROGRAM CSOILP - CALCULATION OF AT-REST PRESSURES BY COMBINATION
OF "GRAVITY-TURN-ON" AND THEORY OF ELASTICITY COMPONENTS
DATE: 9-AUGUST-06 TIME: 9:22:35

* INPUT DATA *

I.--HEADING
'DOVER DSA
'M5 PMF - DS

II.--SURFACE POINTS

DIST. FROM WALL (FT)	ELEVATION (FT)
0.0	902.0

III.--SOIL LAYER DATA

<WEIGHT SAT.	(PCF)> MST.	INTERNAL (DEG)	ELASTICITY COEFF.	BOTTOM ELEV. (FT)
125.0	120.0	33.0	0.00	883.0
128.0	125.0	0.0	0.00	867.0

IV.--WATER DATA

UNIT WEIGHT:	62.5 (PCF)
ELEVATION:	907.5 (FT)

Analysis Results

PROGRAM CSOILP - CALCULATION OF AT-REST PRESSURES BY COMBINATION
OF "GRAVITY-TURN-ON" AND THEORY OF ELASTICITY COMPONENTS
DATE: 9-AUGUST-06 TIME: 9:21:45

* RESULTS *

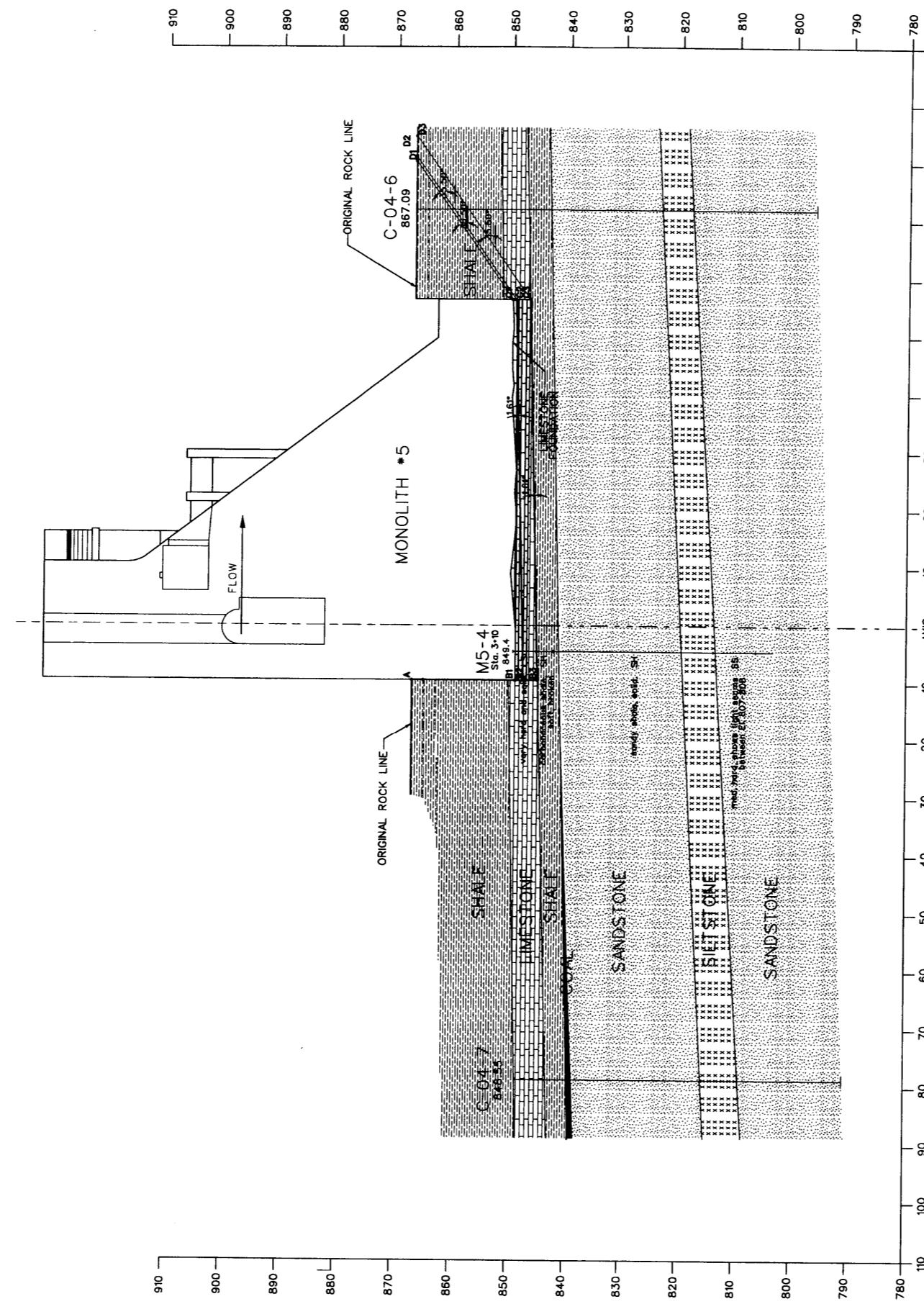
I.--HEADING
'DOVER DSA
'M5 PMF - DS

II.--EQUIVALENT SURCHARGE LOAD DUE TO IRREGULAR SURFACE ABOVE 902.0 (FT)
NONE

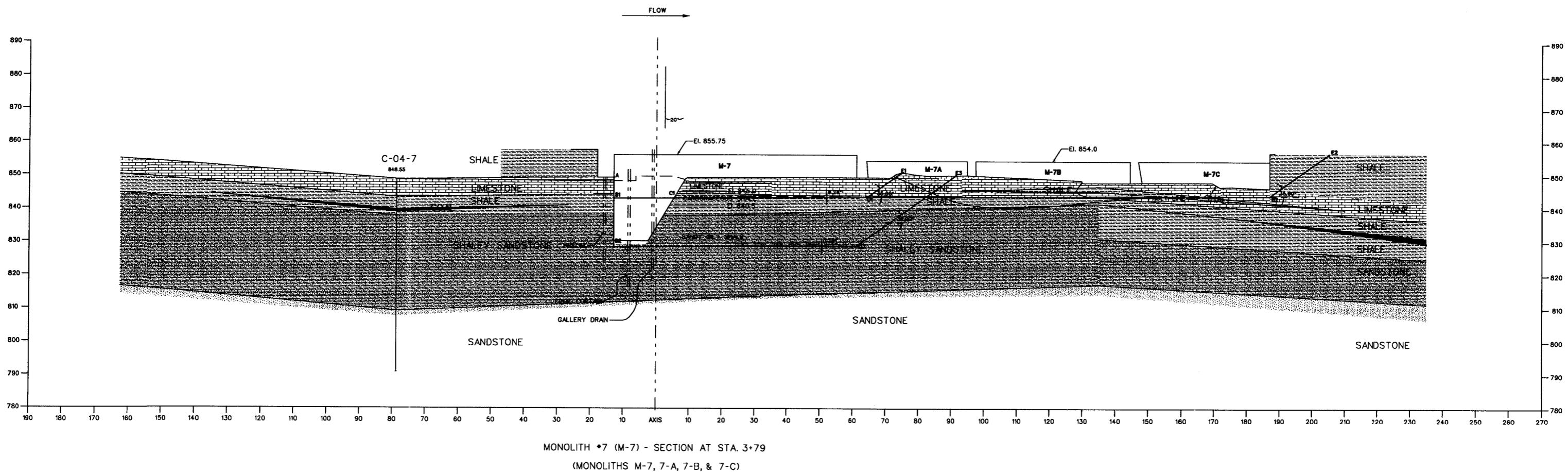
III.--PRESSURES ON WALL BELOW EL. 902.0 (FT)

ELEVATION (FT)	GRAVITY- TURN-ON PRESSURE (PSF)	ELASTICITY COMPONENT PRESSURE (PSF)	COMBINED GTO & ELASTICITY PRESSURE (PSF)
902.00	0.00	0.00	0.00
901.00	28.46	0.00	28.46
900.00	56.92	0.00	56.92
899.00	85.38	0.00	85.38
898.00	113.84	0.00	113.84
897.00	142.30	0.00	142.30
896.00	170.76	0.00	170.76
895.00	199.22	0.00	199.22
894.00	227.68	0.00	227.68
893.00	256.14	0.00	256.14
892.00	284.60	0.00	284.60
891.00	313.06	0.00	313.06
890.00	341.52	0.00	341.52
889.00	369.98	0.00	369.98
888.00	398.44	0.00	398.44
887.00	426.90	0.00	426.90
886.00	455.36	0.00	455.36
885.00	483.82	0.00	483.82
884.00	512.28	0.00	512.28
883.00+	540.74	0.00	540.74
883.00-	1187.50	0.00	1187.50
882.00	1253.00	0.00	1253.00
881.00	1318.50	0.00	1318.50
880.00	1384.00	0.00	1384.00
879.00	1449.50	0.00	1449.50
878.00	1515.00	0.00	1515.00
877.00	1580.50	0.00	1580.50
876.00	1646.00	0.00	1646.00
875.00	1711.50	0.00	1711.50
874.00	1777.00	0.00	1777.00
873.00	1842.50	0.00	1842.50
872.00	1908.00	0.00	1908.00
871.00	1973.50	0.00	1973.50
870.00	2039.00	0.00	2039.00
869.00	2104.50	0.00	2104.50
868.00	2170.00	0.00	2170.00
867.00+	2235.50	0.00	2235.50

Resultant Force for Combined Pressures = 32521.04 lbs
Resultant Location at elev. = 876.58 feet



FAILURE PLANE DETAILS		SLIDING FAILURE MODES		FAILURE PLANE FOLLOWS SEGMENTS:		
NOTES:		Deep-seated sliding on xxxxxxxx		A, B1, C1, D1, E1 & A, B2, D2, E2		
1. xxxxxxxxx						
SEGMENTS	SEGMENT DEFINITION	PHI ANGLE (DEG)	COHESION (PSI)	SHEAR STRENGTH	ANGLE (DEG)	PASSIVE WEDGE ANGLE (DEG)
A-B1, A-B2	Tension Crack	NA	NA	NA	NA	NA
B1-C1	Dam Key (Reinforced Concrete)	36	7	NA	NA	NA
C1-D1, D1-D2	Shale	19	0.5	NA	NA	NA
D1-E1, D2-E2, D3-E3	Fault (as described in foundation report)	19	0	NA	NA	NA
B2-D2	Shaleey Sandstone	29	2.5	NA	NA	NA



(CORRELATING ROCK STRATA USING 2004, 1983, AND 1935 BORINGS)

Monolith 7

Case 1: Spillway Crest

Head water	EL 916.0
Tail water	EL 868.0
Apron F.S. Sliding	1.71
Base in Compression	100 %
F.S. Sliding	1.62
F.S. Bearing	6.35

Case 2: 300 year event

Head water	EL 918.3
Tail water	EL 881.4
Apron F.S. Sliding	1.41
Base in Compression	100 %
F.S. Sliding	1.88
F.S. Bearing	6.51

Case 3: PMF

Head water	EL 937.67
Tail water	EL 907.48
Apron F.S. Sliding	1.14
Base in Compression	100 %
F.S. Sliding	1.10
F.S. Bearing	12.16

IFF : See chart and table for various head and tail waters.



FOUNDATION PARAMETERS

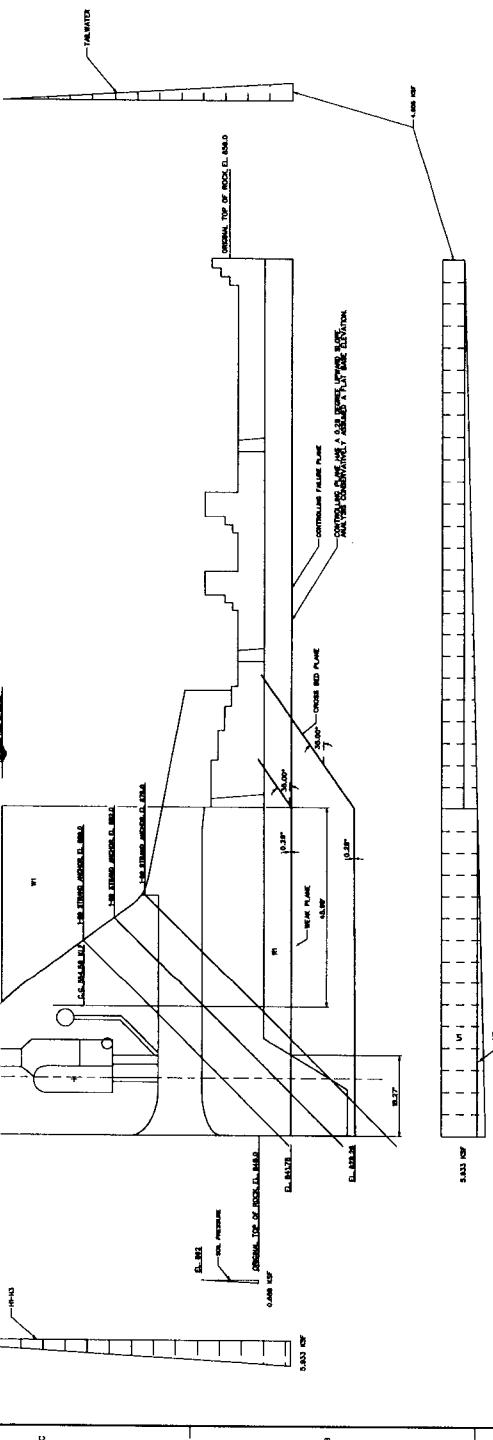
1. 0% DRAIN EFFECTIVENESS
2. ALONG WEAK PLANE: PHI = 9.5, C = 0.25 PSI
(REDUCED DUE TO STRAIN COMPATIBILITY)
3. WEAK PLANE ANGLE 1 = 0 DEGREES
4. ALONG WEAK PLANE 2 PHI = 28, C = 2.5 PSI
5. ALONG CROSSED PLANE PHI = 19, C = 0 PSI
6. WEAK PLANE ANGLE 2 = 0.28
7. ROCK WEDGE FAILURE ANGLE = 35.0
8. ALLOWABLE BEARING CAPACITY = 336 PSI = 52.7 KSF

F.S. SLIDING = 1.10

BASE IN COMPRESSION = 100%

COMPUTED WITH APPROXIMATE RESULTS IN THE FOLLOWING:

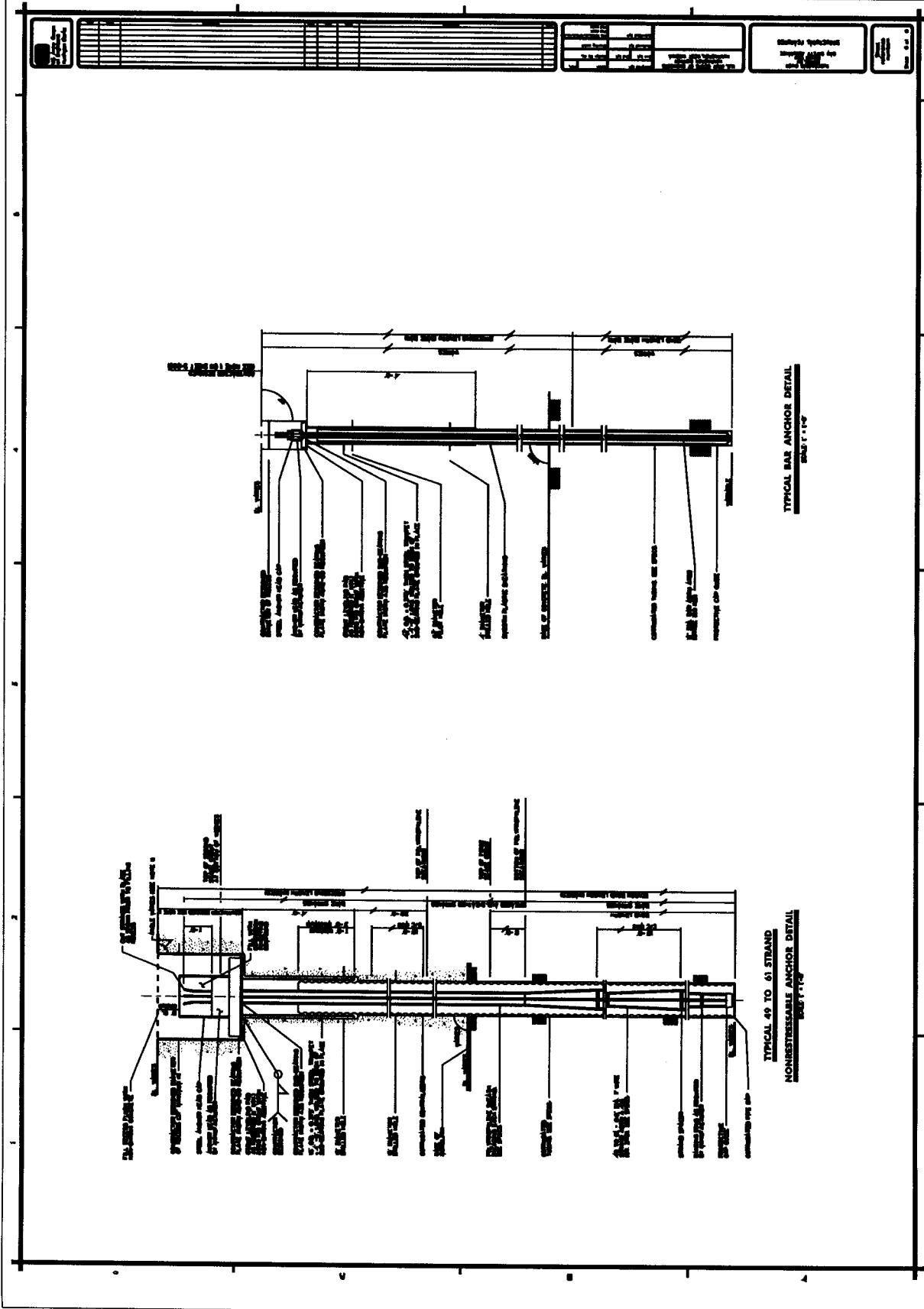
LOAD WATER POOL G. 0.5747

EARTH PRESSURE
0.522 KSF
0.522 KSF
0.522 KSF

FOUNDATION PRESSURE

MONOLITH 7
STA. 3-179
LOAD CASE 3: PMF

4



INPUTS	Var. Name	Notes
Head Water Elevation	916	HW
Tail Water Elevation	868	TW
Failure Plane 1 Elevation	841.75	BaseEI
Failure Plane 1 Angle	0	BaseS
Toe Elevation 1	841.75	ToeEL
Failure Plane 2 Elevation	828.26	BaseEI2
Failure Plane 2 Angle	0.28	BaseS2
Toe Elevation 1	828.63	ToeEL2
Drain Efficiency	0%	DrainEff
Top Elevation	916	TopEl
Base Width	75.25	BaseW
Length of Monolith	34	MonLength
Analysis Length	1	Length
Backfill Elevation U/S	860	BFUS
Backfill Elevation D/S	841.75	BFDS
Backfill Ko	1	Ko
Backfill Gamma moist	0.11	gamma_m in kcf
Backfill Gamma Sat.	0.115	gamma_sat in kcf
Failure Plane 1 Cohesior	0.25	BaseC in psi
Failure Plane 1 phi	9.50	BasePhi
Failure Plane 2 Cohesior	2.50	BaseC2 in psi
Failure Plane 2 phi	29.00	BasePhi2
Crossbed Cohesion	0.00	XBedC
Crossbed Phi	19.00	XBedPhi
Crossbed Failure Angle	35.00	XBedAng
Gamma Rock	0.169	gamma_r
Top Rock US	849.00	RockUS Must be greater than or equal to Base Elevation
Top Rock DS	841.75	RockDS Must be greater than or equal to Base Elevation
Bearing Capacity	52.7	BearCap in ksf
Row 1 Anchors		
# of Anchors per Mono	1	
# of Strands per Anchor	59	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	45.0	Angle down from horizontal in degrees
Elevation of Anchors	889.0	AnchEl Elevation Anchors are Installed
Distance from Toe	39.4	AnchLoc Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	43.15	V_Anch Calculated in Kips per Length
Horizontal Anchor Force	43.15	H_Anch Calculated in Kips per Length
Row 2 Anchors		
# of Anchors per Mono	1	
# of Strands per Anchor	59	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	45.0	Angle down from horizontal in degrees
Elevation of Anchors	882.0	AnchEl2 Elevation Anchors are Installed
Distance from Toe	30.8	AnchLoc2 Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	43.15	V_Anch2 Calculated in Kips per Length
Horizontal Anchor Force	43.15	H_Anch2 Calculated in Kips per Length
Row 3 Anchors		
# of Anchors per Mono	1	
# of Strands per Anchor	59	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	45.0	Angle down from horizontal in degrees
Elevation of Anchors	875.0	AnchEl3 Elevation Anchors are Installed
Distance from Toe	22.1	AnchLoc3 Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	43.15	V_Anch3 Calculated in Kips per Length
Horizontal Anchor Force	43.15	H_Anch3 Calculated in Kips per Length



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

**Monolith 7 - Spillway
Deducts**

Computed By: SAW Date 28 Nov 2006

Checked By: ECV Date 28 Nov 2006

Item	Width	Height	Length	Volume	Arm	Moment
Gallery - Rect. Portion	6.00 X	15.00 X	34.00 X	3060.00 X	62.75 =	192015.0
Gallery - Arch Portion	6.00 X	3.00 X	34.00 X	480.66 X	62.75 =	30161.6
Sluice Gate Mach. Recess	6.00 X	18.58 X	18.00 X	2007.00 X	56.75 =	113897.3
Sluice Gate Mach. Recess	8.25 X	2.25 X	18.00 X	334.13 X	56.75 =	18961.6
Sluice Gate Shaft Ext.	4.00 X	8.40 X	2.00 X	211.01 X	56.75 =	11974.8
Pipe Conduit	2.50 X	2.50 X	16.00 X	78.54 X	54.65 =	4291.9
Air Inlet	4.00 X	4.00 X	34.00 X	427.26 X	48.75 =	20828.8
Bulkhead Slot	2.00 X	10.25 X	14.00 X	287.00 X	63 =	18081.0
Sluice Gate Slot	2.00 X	9.25 X	13.00 X	240.50 X	58.25 =	14009.1
Sluice	75.25 X	10.00 X	10.00 X	7525.00 X	38.066 =	286446.7
				<u>14651.10</u>		<u>710667.7</u>
Equivalent Square Deduct	20.76 X	20.76 X	34 X	14651.10 X	48.51 =	710667.7
Item	Width	Height	Length	Volume	Arm	Moment
Rock - Rect. Portion	55.75 X	6.25 X	34.00 X	11846.88 X	27.88 =	330231.6
Rock - Tri. Portion	3.13 X	6.25 X	34.00 X	332.03 X	56.79 =	18856.6
				<u>12178.91</u>		<u>349088.2</u>
Equivalent Square Rock	57.31 X	6.25 X	34.00 X	12178.91 X	28.66 =	349088.2



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 7 - Spillway
Load Case 1 Analysis Stability Sheet*

Computed By: SAW Date 28 Nov 2006
Checked By: ECV, RSR Date 28 Nov 2006

	Width	X	Height	X	Unit Wt.	X	Length	Vertical	Horizontal	Arm	Moments
CONCRETE											
C1			Dam Concrete from Microstation					554.58		45.990	25,505.13
C2	20.76 X	20.76 X	0.15 X	1 X	-1			(64.64)		48.506	(3,135.30)
			Concrete Subtotal =					489.94			22,369.84
Anchor Forces											
AV1	1 - 59 Strand Anchors @ 45 degrees							43.15		39.375	1,699.23
AH1	1 - 59 Strand Anchors @ 45 degrees								43.15	47.250	2,039.07
AV2	1 - 59 Strand Anchors @ 45 degrees							43.15		30.750	1,327.01
AH2	1 - 59 Strand Anchors @ 45 degrees								43.15	40.250	1,736.99
AV3	1 - 59 Strand Anchors @ 45 degrees							43.15		22.125	954.80
AH3	1 - 59 Strand Anchors @ 45 degrees								43.15	33.250	1,434.90
MISC. VERTICAL											
W1	75.25 X	6 X	0.063 X	0.2941 X	1			8.30		38.066	315.93
CV	Net Vertical Crest Pressure & Moment							0.00			0.00
R1	57.31 X	6.25 X	0.169 X	1 X	1			60.36		28.663	1,730.04
UPLIFT											
U1	75.25 X	56.19 X	0.0625 X	1 X	-1			(264.27)		37.625	(9,943.11)
U2	62.75 X	15.06 X	0.0625 X	1 X	-0.5			(29.53)		41.833	(1,235.41)
U3	0 X	18.06 X	0.0625 X	1 X	-1			0.00		75.250	0.00
U4	12.5 X	3 X	0.0625 X	1 X	-0.5			(1.17)		71.083	(83.30)
U5	12.5 X	15.06 X	0.0625 X	1 X	-1			(11.77)		69.000	(811.83)
U6	0 X	56.19 X	0.0625 X	1 X	1				0.00	0.000	0.00
U7	0 X	15.06 X	0.0625 X	1 X	0.5				0.00	0.000	0.00
U8	0 X	18.06 X	0.0625 X	1 X	1				0.00	0.000	0.00
U9	0 X	3 X	0.0625 X	1 X	0.5				0.00	0.000	0.00
U10	0 X	15.06 X	0.0625 X	1 X	1				0.00	0.000	0.00
HYDROSTATIC											
H1	74.25 X	74.25 X	0.0625 X	1 X	-0.5				(172.28)	24.750	(4,264.01)
H2	24.85 X	24.85 X	0.0625 X	1 X	0.5				19.30	8.283	159.85
H3	0.0 X	0.0 X	0.0625 X	1 X	0.5				0.00	68.250	0.00
MISC. HORIZONTAL											
E1	11 X	11 X	0.053 X	1 X	-0.5				(3.18)	10.917	(34.67)
CH	Net Horizontal Crest Pressure							0.00			
Sum V <u>381.33</u> Sum H <u>(26.70)</u>									Sum M		<u>17,395.33</u>

$$M/V = 45.62 \text{ ft.}$$

$$e = M/V-B/2 = 7.99 \text{ ft.}$$

%Base in Compression = 100.0%

Min. Found. Pressure= 1.8379 ksf
Max. Found. Pressure= 8.297 ksf
Bearing Capacity= 52.7 ksf

Sliding F.S. = 1.62
Bearing F.S. = 6.35



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 7 - Spillway
Apron Floatation and Sliding*

Computed By: SAW Date 28 Nov 2006
Checked By: ECV, RSR Date 28 Nov 2006

Apron Length = 102.00 Water Velocity Impacting Baffles = 35
Apron Phi = 9.5
Apron Cohesion = 0.25

Item	Width	Height	Length	Unit Wt.	Weight
Upstream Section					
Step 1	10.63 X	18.25 X	102.00 X	0.150 =	2966.77
Step 2	6.79 X	16.75 X	102.00 X	0.150 =	1740.53
Step 3	5.42 X	15.25 X	102.00 X	0.150 =	1263.84
Step 4	4.67 X	13.75 X	102.00 X	0.150 =	981.75
Step 5	5.17 X	12.25 X	102.00 X	0.150 =	968.36
Training Walls	26.50 X	10.00 X	20.62 X	0.150 =	819.49
Center Section					
Base Slab	49.58 X	12.25 X	102.00 X	0.150 =	9293.16
Baffle Step 1	1.75 X	1.25 X	86.25 X	0.150 =	28.30
Baffle Step 2	1.75 X	3.00 X	86.25 X	0.150 =	67.92
Baffle Step 3	5.50 X	7.50 X	86.25 X	0.150 =	533.67
Downstream Section					
Base Slab	42.50 X	12.25 X	102.00 X	0.150 =	7965.56
End Sill Step 1	2.00 X	2.00 X	102.00 X	0.150 =	61.20
End Sill Step 2	2.00 X	4.00 X	102.00 X	0.150 =	122.40
End Sill Step 3	2.00 X	6.00 X	102.00 X	0.150 =	183.60
Water Pressures					
90% or 60% TW	124.75 X	12.60 X	102.00 X	0.0625 =	10020.54
Uplift Pressures					
U1	124.75 X	26.25 X	102.00 X	-0.0625 =	-20876.13
U2	124.75 X	29.94 X	102.00 X	-0.0625 =	-11905.36
Apron Anchors					
	42 Anchors	X	282.74 kips/anchor	=	11875.08
				Sum V =	16110.69
				Lateral Load on Baffles =	9.42
				Floatation Factor of Safety =	1.71



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

**Monolith 7 - Spillway
Crest Pressures**

Computed By: SAW Date 28 Nov 2006
Checked By: ECV Date 28 Nov 2006

Pool Elevation	Net Horizontal Force	Net Vertical Force	Net Moment
916	-1.125	1.129	2.589
917	-1.169	2.287	67.497
918	-1.213	3.444	132.405
919	-1.258	4.601	197.313
920	-1.302	5.759	262.221
921	-1.346	6.916	327.129
922	-1.390	8.074	392.037
923	-1.435	9.231	456.945
924	-1.479	10.388	521.853
Calculated from Sap2000	924.5	-1.501	554.307
	925	-1.705	535.442
	926	-2.112	497.710
	927	-2.519	459.978
	928	-2.927	422.246
	929	-3.334	384.515
	930	-3.741	346.783
	931	-4.149	309.051
	932	-4.556	271.319
Calculated from Sap2000	933	-4.964	233.588
	934	-5.389	159.106
	935	-5.815	84.625
Calculated from Sap2000	935.89	-6.195	18.337
	936	-6.239	7.064
	937	-6.643	-95.418
	938	-7.048	-197.901
Calculated from Sap2000	938.61	-7.295	-260.415
	939	-7.458	-313.351
	940	-7.877	-449.083
	941	-8.296	-584.816
Calculated from Sap2000	941.50	-8.505	-652.682



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 7 - Spillway
Load Case 1 Analysis at Failure Plane 1*

Computed By: SAW Date 28 Nov 2006
Checked By: ECV, RSR Date 28 Nov 2006

Key Concrete

Length	16.375 ft	Vertical Force in Key =	41.604
Length in Compression	16.375 ft	Concrete Phi =	38.000
U/S Found. Pressure	1.838 ksf		
D/S Pressure	3.243 ksf		

Key Reinforcement

Number of Bars Intersected	4.00		
Size of Bars	0.75 in		
Spacing of Bars	2.50 ft	Reinforcing Resisting Force =	9.90
Area of Bars per Ft	0.71 in ²	Equivalent Cohesion =	4.20
Shear Strength of Bars	14.00 ksi		

Shale Bedding Plane

Length	58.875 ft	Vertical Force on Shale =	339.723
Length in Compression	58.875 ft	Shale Phi =	9.50
U/S Found. Pressure	3.243 ksf	Shale Cohesion =	0.25
D/S Pressure	8.297 ksf		
		Wsk =	41.60
		Wss =	339.72
		Vs =	0.00
		Us =	0.00
		(HI-Hr)s =	26.70
		Ps =	1.344

Apron

Wa =	264.67
Va =	214.66
Ua =	321.39
(HI-Hr)a =	9.42
Pa =	-1.344

Passive Wedge

Wp	0.00 X	0.00 X	0.162 X	0.5 =	0.00
Vp	0.00 X	26.25 X	0.0625 X	1 =	0.00
	0.00 X	26.25 X	0.0625 X	1 =	0.00
	0.00 X	0.00 X	0.0625 X	0.5 =	0.00
				Up =	0.00
				(HI-Hr)p =	-2.24
				Pp =	0.000

Sum Horizontal Forces = 0.00

Sliding Factor of Safety = 3.83



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

**Monolith 7 - Spillway
Load Case 1 Analysis at Failure Plane 2**

Computed By:	SAW	Date	28 Nov 2006
Checked By:	ECV	Date	28 Nov 2006

Structure Wedge	Width	Height	Length	Unit Wt.	Weight
Key Concrete		231.07 X	1.00 X	0.150 =	34.66
Shale		217.50 X	1.00 X	0.162 =	35.15
Sandstone		754.66 X	1.00 X	0.148 =	111.69
				Ws =	181.50
External Vertical Loads		Sum V from Failure Plane 1 Analysis - uplift			
Uplift 1	75.25 X	69.31 X	1.00 X	0.0625 =	325.99
Uplift 2	75.25 X	18.43 X	1.00 X	0.0625 =	43.33
				Us =	369.32
Hydrostatic Left	87.74 X	87.74 X	1.00 X	0.0625 =	240.57
Hydrostatic Right	39.37 X	39.37 X	1.00 X	-0.0625 =	-48.44
Silt Load		E1 from Failure Plane 1 Analysis			
Horizontal Uplift on Plane 1		From Failure Plane 1 Analysis			
				=	3.18
				=	0.00
				(Hi-Hr)s =	195.31
				Ps =	-3.260

Passive Wedge (Rock)					
Shale		82.72 X	1.00 X	0.162 =	13.37
Sandstone		123.62 X	1.00 X	0.148 =	18.30
				Wr =	31.66
Vertical Load from Water	18.74 X	26.25 X	1.00 X	0.0625 =	30.75
Vertical Load from Apron				=	44.45
				Vr =	75.20
Uplift 1	22.88 X	48.99 X	1.00 X	0.0625 =	70.05
Uplift 2	22.88 X	20.32 X	1.00 X	0.0625 =	14.53
				Ur =	84.58
Hydrostatic Left	39.37 X	39.37 X	1.00 X	0.0625 =	48.44
Hydrostatic Left	26.25 X	26.25 X	1.00 X	-0.0625 =	-21.53
				(Hi-Hr)r =	26.91
				Pr =	0.000

Passive Wedge (Apron)					
Concrete	17.49 X	12.25 X	1.00 X	0.1500 =	16.07
				Wa =	16.07
Vertical Load from Water	17.49 X	14.00 X	1.00 X	0.0625 =	15.31
				Va =	15.31
Uplift 1	21.36 X	14.00 X	1.00 X	0.0625 =	18.69
Uplift 2	21.36 X	34.99 X	1.00 X	0.0625 =	23.35
				Ua =	42.04
Hydrostatic Left	26.25 X	26.25 X	1.00 X	0.0625 =	21.53
Hydrostatic Left	14.00 X	14.00 X	1.00 X	-0.0625 =	-6.13
				(Hi-Hr)a =	15.41
				Pa =	3.260

Ps + Pp = 0.00

Sliding Factor of Safety = 1.62

INPUTS	Var. Name	Notes
Head Water Elevation	918.3	HW
Tail Water Elevation	881.4	TW
Failure Plane 1 Elevation	841.75	BaseEl
Failure Plane 1 Angle	0	BaseS
Toe Elevation 1	841.75	ToeEL
Failure Plane 2 Elevation	828.26	BaseEl2
Failure Plane 2 Angle	0.28	BaseS2
Toe Elevation 1	828.63	ToeEL2
Drain Efficiency	0%	DrainEff
Top Elevation	916	TopEl
Base Width	75.25	BaseW
Length of Monolith	34	MonLength
Analysis Length	1	Length
Backfill Elevation U/S	860	BFUS
Backfill Elevation D/S	841.75	BFDS
Backfill Ko	1	Ko
Backfill Gamma moist	0.11	gamma_m in kcf
Backfill Gamma Sat.	0.115	gamma_sat in kcf
Failure Plane 1 Cohesion	0.25	BaseC in psi
Failure Plane 1 phi	9.50	BasePhi
Failure Plane 2 Cohesion	2.50	BaseC2 in psi
Failure Plane 2 phi	29.00	BasePhi2
Crossbed Cohesion	0.00	XBedC
Crossbed Phi	19.00	XBedPhi
Crossbed Failure Angle	35.00	XBedAng
Gamma Rock	0.169	gamma_r
Top Rock US	849.00	RockUS Must be greater than or equal to Base Elevation
Top Rock DS	841.75	RockDS Must be greater than or equal to Base Elevation
Bearing Capacity	52.7	BearCap in ksf
Row 1 Anchors		
# of Anchors per Mono	1	
# of Strands per Anchor	59	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	45.0	Angle down from horizontal in degrees
Elevation of Anchors	889.0	AnchEl Elevation Anchors are Installed
Distance from Toe	39.4	AnchLoc Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	43.15	V_Anch Calculated in Kips per Length
Horizontal Anchor Force	43.15	H_Anch Calculated in Kips per Length
Row 2 Anchors		
# of Anchors per Mono	1	
# of Strands per Anchor	59	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	45.0	Angle down from horizontal in degrees
Elevation of Anchors	882.0	AnchEl2 Elevation Anchors are Installed
Distance from Toe	30.8	AnchLoc2 Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	43.15	V_Anch2 Calculated in Kips per Length
Horizontal Anchor Force	43.15	H_Anch2 Calculated in Kips per Length
Row 3 Anchors		
# of Anchors per Mono	1	
# of Strands per Anchor	59	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	45.0	Angle down from horizontal in degrees
Elevation of Anchors	875.0	AnchEl3 Elevation Anchors are Installed
Distance from Toe	22.1	AnchLoc3 Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	43.15	V_Anch3 Calculated in Kips per Length
Horizontal Anchor Force	43.15	H_Anch3 Calculated in Kips per Length



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

**Monolith 7 - Spillway
Deducts**

Computed By: SAW Date 28 Nov 2006

Checked By: ECV Date 28 Nov 2006

Item	Width	Height	Length	Volume	Arm	Moment
Gallery - Rect. Portion	6.00 X	15.00 X	34.00 X	3060.00 X	62.75 =	192015.0
Gallery - Arch Portion	6.00 X	3.00 X	34.00 X	480.66 X	62.75 =	30161.6
Sluice Gate Mach. Recess	6.00 X	18.58 X	18.00 X	2007.00 X	56.75 =	113897.3
Sluice Gate Mach. Recess	8.25 X	2.25 X	18.00 X	334.13 X	56.75 =	18961.6
Sluice Gate Shaft Ext.	4.00 X	8.40 X	2.00 X	211.01 X	56.75 =	11974.8
Pipe Conduit	2.50 X	2.50 X	16.00 X	78.54 X	54.65 =	4291.9
Air Inlet	4.00 X	4.00 X	34.00 X	427.26 X	48.75 =	20828.8
Bulkhead Slot	2.00 X	10.25 X	14.00 X	287.00 X	63 =	18081.0
Sluice Gate Slot	2.00 X	9.25 X	13.00 X	240.50 X	58.25 =	14009.1
Sluice	75.25 X	10.00 X	10.00 X	7525.00 X	38.066 =	286446.7
				<u>14651.10</u>		<u>710667.7</u>
Equivalent Square Duct	20.76 X	20.76 X	34 X	14651.10 X	48.51 =	710667.7
Item	Width	Height	Length	Volume	Arm	Moment
Rock - Rect. Portion	55.75 X	6.25 X	34.00 X	11846.88 X	27.88 =	330231.6
Rock - Tri. Portion	3.13 X	6.25 X	34.00 X	332.03 X	56.79 =	18856.6
				<u>12178.91</u>		<u>349088.2</u>
Equivalent Square Rock	57.31 X	6.25 X	34.00 X	12178.91 X	28.66 =	349088.2



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 7 - Spillway
Load Case 2 Analysis Stability Sheet*

Computed By:	<u>SAW</u>	Date	28 Nov 2006
Checked By:	<u>ECV, RSR</u>	Date	28 Nov 2006

	Width	X	Height	X	Unit Wt.	X	Length	Vertical	Horizontal	Arm	Moments
CONCRETE											
C1	20.76	X	20.76	X	0.15	X	1	554.58		45.990	25,505.13
C2							-1	(64.64)		48.506	(3,135.30)
Concrete Subtotal =											
								489.94			22,369.84
Anchor Forces											
AV1	1	- 59 Strand Anchors @ 45 degrees						43.15		39.375	1,699.23
AH1	1	- 59 Strand Anchors @ 45 degrees							43.15	47.250	2,039.07
AV2	1	- 59 Strand Anchors @ 45 degrees						43.15		30.750	1,327.01
AH2	1	- 59 Strand Anchors @ 45 degrees							43.15	40.250	1,736.99
AV3	1	- 59 Strand Anchors @ 45 degrees						43.15		22.125	954.80
AH3	1	- 59 Strand Anchors @ 45 degrees							43.15	33.250	1,434.90
MISC. VERTICAL											
W1	75.25	X	10	X	0.063	X	0.2941	13.83		38.066	526.56
CV								3.44			132.40
R1	57.31	X	6.25	X	0.169	X	1	60.36		28.663	1,730.04
UPLIFT											
U1	75.25	X	62.666	X	0.0625	X	1	1 X -1	(294.73)	37.625	(11,089.13)
U2	62.75	X	11.577	X	0.0625	X	1	1 X -0.5	(22.70)	41.833	(949.72)
U3	0	X	13.884	X	0.0625	X	1	1 X -1	0.00	75.250	0.00
U4	12.5	X	2.3063	X	0.0625	X	1	1 X -0.5	(0.90)	71.083	(64.04)
U5	12.5	X	11.577	X	0.0625	X	1	1 X -1	(9.04)	69.000	(624.09)
U6	0	X	62.666	X	0.0625	X	1	1 X 1		0.00	0.00
U7	0	X	11.577	X	0.0625	X	1	1 X 0.5		0.00	0.00
U8	0	X	13.884	X	0.0625	X	1	1 X 1		0.00	0.00
U9	0	X	2.3063	X	0.0625	X	1	1 X 0.5		0.00	0.00
U10	0	X	11.577	X	0.0625	X	1	1 X 1		0.00	0.00
HYDROSTATIC											
H1	76.55	X	76.55	X	0.0625	X	1	1 X -0.5		(183.12)	25.517
H2	28.69	X	28.69	X	0.0625	X	1	1 X 0.5		25.72	9.563
H3	8.3	X	8.3	X	0.0625	X	1	1 X 0.5		2.15	71.017
MISC. HORIZONTAL											
E1	11	X	11	X	0.053	X	1	1 X -0.5		(3.18)	10.917
CH										(1.21)	
Net Horizontal Crest Pressure											
Sum V	<u>369.67</u>	Sum H	<u>(30.17)</u>	Sum M	<u>16,915.39</u>						

M/V = 45.76 ft.
e = M/V-B/2 = 8.13 ft.
%Base in Compression = 100.0%

Min. Found. Pressure= 1.7266 ksf
Max. Found. Pressure= 8.0984 ksf
Bearing Capacity= 52.7 ksf

Sliding F.S. = 1.88
Bearing F.S. = 6.51



**US Army Corps
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Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 7 - Spillway
Apron Floatation and Sliding*

Computed By: SAW Date 28 Nov 2006

Checked By: ECV, RSR Date 28 Nov 2006

Apron Length = 102.00 Water Velocity Impacting Baffles = 35
Apron Phi = 9.5
Apron Cohesion = 0.25

Item	Width	Height	Length	Unit Wt.	Weight
Upstream Section					
Step 1	10.63 X	18.25 X	102.00 X	0.150 =	2966.77
Step 2	6.79 X	16.75 X	102.00 X	0.150 =	1740.53
Step 3	5.42 X	15.25 X	102.00 X	0.150 =	1263.84
Step 4	4.67 X	13.75 X	102.00 X	0.150 =	981.75
Step 5	5.17 X	12.25 X	102.00 X	0.150 =	968.36
Training Walls	26.50 X	10.00 X	20.62 X	0.150 =	819.49
Center Section					
Base Slab	49.58 X	12.25 X	102.00 X	0.150 =	9293.16
Baffle Step 1	1.75 X	1.25 X	86.25 X	0.150 =	28.30
Baffle Step 2	1.75 X	3.00 X	86.25 X	0.150 =	67.92
Baffle Step 3	5.50 X	7.50 X	86.25 X	0.150 =	533.67
Downstream Section					
Base Slab	42.50 X	12.25 X	102.00 X	0.150 =	7965.56
End Sill Step 1	2.00 X	2.00 X	102.00 X	0.150 =	61.20
End Sill Step 2	2.00 X	4.00 X	102.00 X	0.150 =	122.40
End Sill Step 3	2.00 X	6.00 X	102.00 X	0.150 =	183.60
Water Pressures					
90% or 60% TW	124.75 X	16.44 X	102.00 X	0.0625 =	13074.42
Uplift Pressures					
U1	124.75 X	39.65 X	102.00 X	-0.0625 =	-31532.90
U2	124.75 X	23.02 X	102.00 X	-0.0625 =	-9152.25
Apron Anchors					
	42 Anchors	X	282.74 kips/anchor	=	11875.08
				Sum V =	11260.91
				Lateral Load on Baffles =	9.42
				Floatation Factor of Safety =	1.41



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION
Monolith 7 - Spillway
Crest Pressures

Computed By: SAW Date 28 Nov 2006
Checked By: ECV Date 28 Nov 2006

	Pool Elevation	Net Horizontal Force	Net Vertical Force	Net Moment
	916	-1.125	1.129	2.589
	917	-1.169	2.287	67.497
	918	-1.213	3.444	132.405
	919	-1.258	4.601	197.313
	920	-1.302	5.759	262.221
	921	-1.346	6.916	327.129
	922	-1.390	8.074	392.037
	923	-1.435	9.231	456.945
	924	-1.479	10.388	521.853
Calculated from Sap2000	924.5	-1.501	10.967	554.307
	925	-1.705	10.819	535.442
	926	-2.112	10.524	497.710
	927	-2.519	10.228	459.978
	928	-2.927	9.933	422.246
	929	-3.334	9.637	384.515
	930	-3.741	9.342	346.783
	931	-4.149	9.046	309.051
	932	-4.556	8.751	271.319
Calculated from Sap2000	933	-4.964	8.455	233.588
	934	-5.389	7.567	159.106
	935	-5.815	6.680	84.625
Calculated from Sap2000	935.89	-6.195	5.890	18.337
	936	-6.239	5.747	7.064
	937	-6.643	4.450	-95.418
	938	-7.048	3.154	-197.901
Calculated from Sap2000	938.61	-7.295	2.363	-260.415
	939	-7.458	1.648	-313.351
	940	-7.877	-0.185	-449.083
	941	-8.296	-2.018	-584.816
Calculated from Sap2000	941.50	-8.505	-2.935	-652.682



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

**Monolith 7 - Spillway
Load Case 2 Analysis at Failure Plane 1**

Computed By:	SAW	Date	28 Nov 2006
Checked By:	ECV, RSR	Date	28 Nov 2006

Key Concrete

Length	16.375 ft	Vertical Force in Key =	39.626
Length in Compression	16.375 ft	Concrete Phi =	38.000
U/S Found. Pressure	1.727 ksf		
D/S Pressure	3.113 ksf		

Key Reinforcement

Number of Bars Intersected	4.00		
Size of Bars	0.75 in		
Spacing of Bars	2.50 ft	Reinforcing Resisting Force =	9.90
Area of Bars per Ft	0.71 in ²	Equivalent Cohesion =	4.20
Shear Strength of Bars	14.00 ksi		

Shale Bedding Plane

Length	58.875 ft	Vertical Force on Shale =	330.040
Length in Compression	58.875 ft	Shale Phi =	9.50
U/S Found. Pressure	3.113 ksf	Shale Cohesion =	0.25
D/S Pressure	8.098 ksf		
		Wsk =	39.63
		Wss =	330.04
		Vs =	0.00
		Us =	0.00
		(Hi-Hr)s =	30.17
		Ps =	2.191

Apron

Wa =	264.67
Va =	244.60
Ua =	398.87
(Hi-Hr)a =	9.42
Pa =	-2.191

Passive Wedge

Wp	0.00 X	0.00 X	0.162 X	0.5 =	0.00
Vp	0.00 X	39.65 X	0.0625 X	1 =	0.00
	0.00 X	39.65 X	0.0625 X	1 =	0.00
	0.00 X	0.00 X	0.0625 X	0.5 =	0.00
				Up =	0.00
				(Hi-Hr)p =	-23.41
				Pp =	0.000

Sum Horizontal Forces = 0.00

Sliding Factor of Safety = 3.18



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

**Monolith 7 - Spillway
Load Case 2 Analysis at Failure Plane 2**

Computed By: SAW Date 28 Nov 2006
Checked By: ECV Date 28 Nov 2006

Structure Wedge	Width	Height	Length	Unit Wt.	Weight
Key Concrete		231.07 X	1.00 X	0.150 =	34.66
Shale		217.50 X	1.00 X	0.162 =	35.15
Sandstone		754.66 X	1.00 X	0.148 =	111.69
				Ws =	181.50
External Vertical Loads		Sum V from Failure Plane 1 Analysis - uplift		Vs =	697.04
Uplift 1	75.25 X	75.79 X	1.00 X	0.0625 =	356.45
Uplift 2	75.25 X	14.25 X	1.00 X	0.0625 =	33.51
				Us =	389.96
Hydrostatic Left	90.04 X	90.04 X	1.00 X	0.0625 =	253.35
Hydrostatic Right	52.77 X	52.77 X	1.00 X	-0.0625 =	-87.03
Silt Load		E1 from Failure Plane 1 Analysis		=	3.18
Horizontal Uplift on Plane 1		From Failure Plane 1 Analysis		=	0.00
				(HI-Hr)s =	169.50
				Ps =	-6.407

Passive Wedge (Rock)					
Shale		82.72 X	1.00 X	0.162 =	13.37
Sandstone		123.62 X	1.00 X	0.148 =	18.30
				Wr =	31.66
Vertical Load from Water	18.74 X	39.65 X	1.00 X	0.0625 =	46.44
Vertical Load from Apron				=	44.45
				Vr =	90.89
Uplift 1	22.88 X	57.13 X	1.00 X	0.0625 =	81.69
Uplift 2	22.88 X	18.66 X	1.00 X	0.0625 =	13.34
				Ur =	95.03
Hydrostatic Left	52.77 X	52.77 X	1.00 X	0.0625 =	87.03
Hydrostatic Left	39.65 X	39.65 X	1.00 X	-0.0625 =	-49.13
				(HI-Hr)r =	37.90
				Pr =	0.000

Passive Wedge (Apron)					
Concrete	17.49 X	12.25 X	1.00 X	0.1500 =	16.07
				Wa =	16.07
Vertical Load from Water	17.49 X	27.40 X	1.00 X	0.0625 =	29.96
				Va =	29.96
Uplift 1	21.36 X	27.40 X	1.00 X	0.0625 =	36.57
Uplift 2	21.36 X	29.73 X	1.00 X	0.0625 =	19.84
				Ua =	56.42
Hydrostatic Left	39.65 X	39.65 X	1.00 X	0.0625 =	49.13
Hydrostatic Left	27.40 X	27.40 X	1.00 X	-0.0625 =	-23.46
				(HI-Hr)a =	25.67
				Pa =	6.407

Ps + Pp = 0.00

Sliding Factor of Safety = 1.88

INPUTS	Var. Name	Notes
Head Water Elevation	937.67	HW
Tail Water Elevation	907.48	TW
Failure Plane 1 Elevation	841.75	BaseEl
Failure Plane 1 Angle	0	BaseS
Toe Elevation 1	841.75	ToeEL
Failure Plane 2 Elevation	828.26	BaseEl2
Failure Plane 2 Angle	0.28	BaseS2
Toe Elevation 1	828.63	ToeEL2
Drain Efficiency	0%	DrainEff
Top Elevation	916	TopEl
Base Width	75.25	BaseW
Length of Monolith	34	MonLength
Analysis Length	1	Length
Backfill Elevation U/S	860	BFUS
Backfill Elevation D/S	841.75	BFDS
Backfill Ko	1	Ko
Backfill Gamma moist	0.11	gamma_m in kcf
Backfill Gamma Sat.	0.115	gamma_sat in kcf
Failure Plane 1 Cohesion	0.25	BaseC in psi
Failure Plane 1 phi	9.50	BasePhi
Failure Plane 2 Cohesion	2.50	BaseC2 in psi
Failure Plane 2 phi	29.00	BasePhi2
Crossbed Cohesion	0.00	XBedC
Crossbed Phi	19.00	XBedPhi
Crossbed Failure Angle	35.00	XBedAng
Gamma Rock	0.169	gamma_r
Top Rock US	849.00	RockUS Must be greater than or equal to Base Elevation
Top Rock DS	841.75	RockDS Must be greater than or equal to Base Elevation
Bearing Capacity	52.7	BearCap in ksf
Row 1 Anchors		
# of Anchors per Mono.	1	
# of Strands per Anchor	59	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	45.0	Angle down from horizontal in degrees
Elevation of Anchors	889.0	AnchEl Elevation Anchors are Installed
Distance from Toe	39.4	AnchLoc Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	43.15	V_Anch Calculated in Kips per Length
Horizontal Anchor Force	43.15	H_Anch Calculated in Kips per Length
Row 2 Anchors		
# of Anchors per Mono.	1	
# of Strands per Anchor	59	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	45.0	Angle down from horizontal in degrees
Elevation of Anchors	882.0	AnchEl2 Elevation Anchors are Installed
Distance from Toe	30.8	AnchLoc2 Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	43.15	V_Anch2 Calculated in Kips per Length
Horizontal Anchor Force	43.15	H_Anch2 Calculated in Kips per Length
Row 3 Anchors		
# of Anchors per Mono.	1	
# of Strands per Anchor	59	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	45.0	Angle down from horizontal in degrees
Elevation of Anchors	875.0	AnchEl3 Elevation Anchors are Installed
Distance from Toe	22.1	AnchLoc3 Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	43.15	V_Anch3 Calculated in Kips per Length
Horizontal Anchor Force	43.15	H_Anch3 Calculated in Kips per Length



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

**Monolith 7 - Spillway
Deducts**

Computed By: SAW Date 28 Nov 2006

Checked By: ECV Date 28 Nov 2006

Item	Width	Height	Length	Volume	Arm	Moment
Gallery - Rect. Portion	6.00 X	15.00 X	34.00 X	3060.00 X	62.75	= 192015.0
Gallery - Arch Portion	6.00 X	3.00 X	34.00 X	480.66 X	62.75	= 30161.6
Sluice Gate Mach. Recess	6.00 X	18.58 X	18.00 X	2007.00 X	56.75	= 113897.3
Sluice Gate Mach. Recess	8.25 X	2.25 X	18.00 X	334.13 X	56.75	= 18961.6
Sluice Gate Shaft Ext.	4.00 X	8.40 X	2.00 X	211.01 X	56.75	= 11974.8
Pipe Conduit	2.50 X	2.50 X	16.00 X	78.54 X	54.65	= 4291.9
Air Inlet	4.00 X	4.00 X	34.00 X	427.26 X	48.75	= 20828.8
Bulkhead Slot	2.00 X	10.25 X	14.00 X	287.00 X	63	= 18081.0
Sluice Gate Slot	2.00 X	9.25 X	13.00 X	240.50 X	58.25	= 14009.1
Sluice	75.25 X	10.00 X	10.00 X	7525.00 X	38.066	= 286446.7
				<u>14651.10</u>		<u>710667.7</u>
Equivalent Square Deduct	20.76 X	20.76 X	34 X	14651.10 X	48.51	= 710667.7
Item	Width	Height	Length	Volume	Arm	Moment
Rock - Rect. Portion	55.75 X	6.25 X	34.00 X	11846.88 X	27.88	= 330231.6
Rock - Tri. Portion	3.13 X	6.25 X	34.00 X	332.03 X	56.79	= 18856.6
				<u>12178.91</u>		<u>349088.2</u>
Equivalent Square Rock	57.31 X	6.25 X	34.00 X	12178.91 X	28.66	= 349088.2



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

**Monolith 7 - Spillway
Load Case 3 Analysis Stability Sheet**

Computed By: SAW Date 28 Nov 2006
Checked By: ECV, RSR Date 28 Nov 2006

	Width	X	Height	X	Unit Wt.	X	Length	Vertical	Horizontal	Arm	Moments
CONCRETE											
C1			Dam Concrete from Microstation					554.58		45.990	25,505.13
C2	20.76 X	20.76 X	0.15 X	1 X	-1			(64.64)		48.506	(3,135.30)
			Concrete Subtotal =					489.94			22,369.84
Anchor Forces											
AV1	1 - 59 Strand Anchors @ 45 degrees							43.15		39.375	1,699.23
AH1	1 - 59 Strand Anchors @ 45 degrees								43.15	47.250	2,039.07
AV2	1 - 59 Strand Anchors @ 45 degrees							43.15		30.750	1,327.01
AH2	1 - 59 Strand Anchors @ 45 degrees								43.15	40.250	1,736.99
AV3	1 - 59 Strand Anchors @ 45 degrees							43.15		22.125	954.80
AH3	1 - 59 Strand Anchors @ 45 degrees								43.15	33.250	1,434.90
MISC. VERTICAL											
W1	75.25 X	10 X	0.063 X	0.2941 X	1			13.83		38.066	526.56
CV	Net Vertical Crest Pressure & Moment							4.45			(95.42)
R1	57.31 X	6.25 X	0.169 X	1 X	1			60.36		28.663	1,730.04
UPLIFT											
U1	75.25 X	84.561 X	0.0625 X	1 X	-1			(397.70)		37.625	(14,963.50)
U2	62.75 X	9.4721 X	0.0625 X	1 X	-0.5			(18.57)		41.833	(777.02)
U3	0 X	11.359 X	0.0625 X	1 X	-1			0.00		75.250	0.00
U4	12.5 X	1.8869 X	0.0625 X	1 X	-0.5			(0.74)		71.083	(52.39)
U5	12.5 X	9.4721 X	0.0625 X	1 X	-1			(7.40)		69.000	(510.61)
U6	0 X	84.561 X	0.0625 X	1 X	1				0.00	0.000	0.00
U7	0 X	9.4721 X	0.0625 X	1 X	0.5				0.00	0.000	0.00
U8	0 X	11.359 X	0.0625 X	1 X	1				0.00	0.000	0.00
U9	0 X	1.8869 X	0.0625 X	1 X	0.5				0.00	0.000	0.00
U10	0 X	9.4721 X	0.0625 X	1 X	1				0.00	0.000	0.00
HYDROSTATIC											
H1	95.92 X	95.92 X	0.0625 X	1 X	-0.5				(287.52)	31.973	(9,192.98)
H2	44.34 X	44.34 X	0.0625 X	1 X	0.5				61.43	14.779	907.94
H3	27.7 X	27.7 X	0.0625 X	1 X	0.5				23.93	77.473	1,853.62
MISC. HORIZONTAL											
E1	11 X	11 X	0.053 X	1 X	-0.5				(3.18)	10.917	(34.67)
CH	Net Horizontal Crest Pressure							(6.64)			
Sum V <u>273.64</u>						Sum H <u>(82.52)</u>		Sum M <u>10,953.40</u>			

$$\begin{aligned} M/V &= 40.03 \text{ ft.} \\ e &= M/V-B/2 = 2.40 \text{ ft.} \\ \% \text{Base in Compression} &= 100.0\% \end{aligned}$$

Min. Found. Pressure = 2.9393 ksf
Max. Found. Pressure = 4.3334 ksf
Bearing Capacity = 52.7 ksf

Sliding F.S. = 1.10
Bearing F.S. = 12.16



**US Army Corps
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Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 7 - Spillway
Apron Floatation and Sliding*

Computed By: SAW Date 28 Nov 2006

Checked By: ECV, RSR Date 28 Nov 2006

Apron Length = 102.00 Water Velocity Impacting Baffles = 35
Apron Phi = 9.5
Apron Cohesion = 0.25

Item	Width	Height	Length	Unit Wt.	Weight
Upstream Section					
Step 1	10.63 X	18.25 X	102.00 X	0.150 =	2966.77
Step 2	6.79 X	16.75 X	102.00 X	0.150 =	1740.53
Step 3	5.42 X	15.25 X	102.00 X	0.150 =	1263.84
Step 4	4.67 X	13.75 X	102.00 X	0.150 =	981.75
Step 5	5.17 X	12.25 X	102.00 X	0.150 =	968.36
Training Walls	26.50 X	10.00 X	20.62 X	0.150 =	819.49
Center Section					
Base Slab	49.58 X	12.25 X	102.00 X	0.150 =	9293.16
Baffle Step 1	1.75 X	1.25 X	86.25 X	0.150 =	28.30
Baffle Step 2	1.75 X	3.00 X	86.25 X	0.150 =	67.92
Baffle Step 3	5.50 X	7.50 X	86.25 X	0.150 =	533.67
Downstream Section					
Base Slab	42.50 X	12.25 X	102.00 X	0.150 =	7965.56
End Sill Step 1	2.00 X	2.00 X	102.00 X	0.150 =	61.20
End Sill Step 2	2.00 X	4.00 X	102.00 X	0.150 =	122.40
End Sill Step 3	2.00 X	6.00 X	102.00 X	0.150 =	183.60
Water Pressures					
90% or 60% TW	124.75 X	32.09 X	102.00 X	0.0625 =	25518.98
Uplift Pressures					
U1	124.75 X	65.73 X	102.00 X	-0.0625 =	-52273.84
U2	124.75 X	18.83 X	102.00 X	-0.0625 =	-7487.98
Apron Anchors					
	42 Anchors	X	282.74 kips/anchor	=	11875.08
				Sum V =	4628.81
				Lateral Load on Baffles =	9.42
				Floatation Factor of Safety =	1.14



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 7 - Spillway
Crest Pressures*

Computed By: SAW Date 28 Nov 2006
Checked By: ECV Date 28 Nov 2006

Pool Elevation	Net Horizontal Force	Net Vertical Force	Net Moment
916	-1.125	1.129	2.589
917	-1.169	2.287	67.497
918	-1.213	3.444	132.405
919	-1.258	4.601	197.313
920	-1.302	5.759	262.221
921	-1.346	6.916	327.129
922	-1.390	8.074	392.037
923	-1.435	9.231	456.945
924	-1.479	10.388	521.853
Calculated from Sap2000	924.5	-1.501	554.307
	925	-1.705	535.442
	926	-2.112	497.710
	927	-2.519	459.978
	928	-2.927	422.246
	929	-3.334	384.515
	930	-3.741	346.783
	931	-4.149	309.051
	932	-4.556	271.319
Calculated from Sap2000	933	-4.964	233.588
	934	-5.389	159.106
	935	-5.815	84.625
Calculated from Sap2000	935.89	-6.195	18.337
	936	-6.239	7.064
	937	-6.643	-95.418
	938	-7.048	-197.901
Calculated from Sap2000	938.61	-7.295	-260.415
	939	-7.458	-313.351
	940	-7.877	-449.083
	941	-8.296	-584.816
Calculated from Sap2000	941.50	-8.505	-652.682



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Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

**Monolith 7 - Spillway
Load Case 3 Analysis at Failure Plane 1**

Computed By: SAW Date 28 Nov 2006

Checked By: ECV, RSR Date 28 Nov 2006

Key Concrete

Length	16.375 ft	Vertical Force in Key =	50.615
Length in Compression	16.375 ft	Concrete Phi =	38.000
U/S Found. Pressure	2.939 ksf		
D/S Pressure	3.243 ksf		

Key Reinforcement

Number of Bars Intersected	4.00		
Size of Bars	0.75 in		
Spacing of Bars	2.50 ft	Reinforcing Resisting Force =	9.90
Area of Bars per Ft	0.71 in ²	Equivalent Cohesion =	4.20
Shear Strength of Bars	14.00 ksi		

Shale Bedding Plane

Length	58.875 ft	Vertical Force on Shale =	223.021
Length in Compression	58.875 ft	Shale Phi =	9.50
U/S Found. Pressure	3.243 ksf	Shale Cohesion =	0.25
D/S Pressure	4.333 ksf		
		Wsk =	50.61
		Wss =	223.02
		Vs =	0.00
		Us =	0.00
		(Hi-Hr)s =	82.52
		Ps =	-1.561

Apron

Wa =	264.67
Va =	366.61
Ua =	585.90
(Hi-Hr)a =	9.42
Pa =	1.561

Passive Wedge

Wp	0.00 X	0.00 X	0.162 X	0.5 =	0.00
Vp	0.00 X	65.73 X	0.0625 X	1 =	0.00
	0.00 X	65.73 X	0.0625 X	1 =	0.00
	0.00 X	0.00 X	0.0625 X	0.5 =	0.00
				Up =	0.00
				(Hi-Hr)p =	-73.58
				Pp =	0.000

Sum Horizontal Forces = 0.00

Sliding Factor of Safety = 1.10



**US Army Corps
of Engineers**
Huntington District

Dover Dam
Tuscarawas River
Dam Safety Assurance

COMPUTATION

Monolith 7 - Spillway
Load Case 3 Analysis at Failure Plane 2

Computed By:	SAW	Date	28 Nov 2006
Checked By:	ECV	Date	28 Nov 2006

Structure Wedge	Width	Height	Length	Unit Wt.	Weight
Key Concrete		231.07 X	1.00 X	0.150 =	34.66
Shale		217.50 X	1.00 X	0.162 =	35.15
Sandstone		754.66 X	1.00 X	0.148 =	111.69
				Ws =	181.50
External Vertical Loads		Sum V from Failure Plane 1 Analysis - uplift		Vs =	698.05
Uplift 1	75.25 X	97.68 X	1.00 X	0.0625 =	459.42
Uplift 2	75.25 X	11.73 X	1.00 X	0.0625 =	27.58
				Us =	487.00
Hydrostatic Left	109.41 X	109.41 X	1.00 X	0.0625 =	374.08
Hydrostatic Right	78.85 X	78.85 X	1.00 X	-0.0625 =	-194.30
Silt Load		E1 from Failure Plane 1 Analysis		=	3.18
Horizontal Uplift on Plane 1		From Failure Plane 1 Analysis		=	0.00
				(Hi-Hr)s =	182.95
				Ps =	-8.592

Passive Wedge (Rock)					
Shale		82.72 X	1.00 X	0.162 =	13.37
Sandstone		123.62 X	1.00 X	0.148 =	18.30
				Wr =	31.66
Vertical Load from Water	18.74 X	65.73 X	1.00 X	0.0625 =	76.99
Vertical Load from Apron				=	44.45
				Vr =	121.44
Uplift 1	22.88 X	80.03 X	1.00 X	0.0625 =	114.44
Uplift 2	22.88 X	17.65 X	1.00 X	0.0625 =	12.62
				Ur =	127.06
Hydrostatic Left	78.85 X	78.85 X	1.00 X	0.0625 =	194.30
Hydrostatic Left	65.73 X	65.73 X	1.00 X	-0.0625 =	-135.01
				(Hi-Hr)r =	59.29
				Pr =	0.000

Passive Wedge (Apron)					
Concrete	17.49 X	12.25 X	1.00 X	0.1500 =	16.07
				Wa =	16.07
Vertical Load from Water	17.49 X	53.48 X	1.00 X	0.0625 =	58.48
				Va =	58.48
Uplift 1	21.36 X	53.48 X	1.00 X	0.0625 =	71.39
Uplift 2	21.36 X	26.55 X	1.00 X	0.0625 =	17.72
				Ua =	89.11
Hydrostatic Left	65.73 X	65.73 X	1.00 X	0.0625 =	135.01
Hydrostatic Left	53.48 X	53.48 X	1.00 X	-0.0625 =	-89.38
				(Hi-Hr)a =	45.64
				Pa =	8.592

Ps + Pp = 0.00

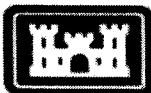
Sliding Factor of Safety = 1.44

INPUTS	Var. Name	Notes
Head Water Elevation	870 HW	
Tail Water Elevation	865 TW	
Failure Plane 1 Elevation	841.75 BaseEl	
Failure Plane 1 Angle	0 BaseS	
Toe Elevation 1	841.75 ToeEL	
Failure Plane 2 Elevation	828.26 BaseEl2	
Failure Plane 2 Angle	0.28 BaseS2	
Toe Elevation 1	828.63 ToeEL2	
Drain Efficiency	0% DrainEff	
Top Elevation	916 TopEl	
Base Width	75.25 BaseW	
Length of Monolith	34 MonLength	
Analysis Length	1 Length	
Backfill Elevation U/S	860 BFUS	
Backfill Elevation D/S	854 BFDS	
Backfill Ko	1 Ko	
Backfill Gamma moist	0.11 gamma_m	in kcf
Backfill Gamma Sat.	0.115 gamma_sat	in kcf
Failure Plane 1 Cohesion	0.25 BaseC	in psi
Failure Plane 1 phi	9.50 BasePhi	
Failure Plane 2 Cohesion	2.50 BaseC2	in psi
Failure Plane 2 phi	29.00 BasePhi2	
Crossbed Cohesion	0.00 XBedC	
Crossbed Phi	19.00 XBedPhi	
Crossbed Failure Angle	35.00 XBedAng	
Gamma Rock	0.169 gamma_r	
Top Rock US	849.00 RockUS	Must be greater than or equal to Base Elevation
Top Rock DS	854.00 RockDS	Must be greater than or equal to Base Elevation
Bearing Capacity	52.7 BearCap	in ksf
Seismic Coefficient	0.10 ac	OBE
Row 1 Anchors		
# of Anchors per Mono.	0	
# of Strands per Anchor	59	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	45.0	Angle down from horizontal in degrees
Elevation of Anchors	889.0 AnchEl	Elevation Anchors are Installed
Distance from Toe	39.4 AnchLoc	Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	0.00 V_Anch	Calculated in Kips per Length
Horizontal Anchor Force	0.00 H_Anch	Calculated in Kips per Length
Row 2 Anchors		
# of Anchors per Mono.	0	
# of Strands per Anchor	59	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	45.0	Angle down from horizontal in degrees
Elevation of Anchors	882.0 AnchEl2	Elevation Anchors are Installed
Distance from Toe	30.8 AnchLoc2	Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	0.00 V_Anch2	Calculated in Kips per Length
Horizontal Anchor Force	0.00 H_Anch2	Calculated in Kips per Length
Row 3 Anchors		
# of Anchors per Mono.	0	
# of Strands per Anchor	59	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	45.0	Angle down from horizontal in degrees
Elevation of Anchors	875.0 AnchEl3	Elevation Anchors are Installed
Distance from Toe	22.1 AnchLoc3	Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	0.00 V_Anch3	Calculated in Kips per Length
Horizontal Anchor Force	0.00 H_Anch3	Calculated in Kips per Length

Dover DSA Monolith 7

899
864

Inputs



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 7 - Spillway
Deducts*

Computed By: SAW Date 01 Jun 2007

Checked By: ECV Date 01 Jun 2007

Item	Width	Height	Length	Volume	Arm	Moment
Gallery - Rect. Portion	6.00 X	15.00 X	34.00 X	3060.00 X	62.75	= 192015.0
Gallery - Arch Portion	6.00 X	3.00 X	34.00 X	480.66 X	62.75	= 30161.6
Sluice Gate Mach. Recess	6.00 X	18.58 X	18.00 X	2007.00 X	56.75	= 113897.3
Sluice Gate Mach. Recess	8.25 X	2.25 X	18.00 X	334.13 X	56.75	= 18961.6
Sluice Gate Shaft Ext.	4.00 X	8.40 X	2.00 X	211.01 X	56.75	= 11974.8
Pipe Conduit	2.50 X	2.50 X	16.00 X	78.54 X	54.65	= 4291.9
Air Inlet	4.00 X	4.00 X	34.00 X	427.26 X	48.75	= 20828.8
Bulkhead Slot	2.00 X	10.25 X	14.00 X	287.00 X	63	= 18081.0
Sluice Gate Slot	2.00 X	9.25 X	13.00 X	240.50 X	58.25	= 14009.1
Sluice	75.25 X	10.00 X	10.00 X	7525.00 X	38.066	= 286446.7
				14651.10		710667.7
Equivalent Square Deduct	20.76 X	20.76 X	34 X	14651.10 X	48.51	= 710667.7
Item	Width	Height	Length	Volume	Arm	Moment
Rock - Rect. Portion	55.75 X	6.25 X	34.00 X	11846.88 X	27.88	= 330231.6
Rock - Tri. Portion	3.13 X	6.25 X	34.00 X	332.03 X	56.79	= 18856.6
				12178.91		349088.2
Equivalent Square Rock	57.31 X	6.25 X	34.00 X	12178.91 X	28.66	= 349088.2



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

**Monolith 7 - Spillway
Seismic Analysis - OBE**

Computed By:	SAW	Date	01 Jun 2007
Checked By:	ECV, RSR	Date	01 Jun 2007

Width	X	Height	X	Unit Wt.	X	Length	Vertical	Horizontal	Arm	Moments						
CONCRETE																
C1		Dam Concrete from Microstation					554.58		45.990	25,505.13						
C2	20.76 X	20.76 X	0.15 X	1 X	-1		(64.64)		48.506	(3,135.30)						
Concrete Subtotal =																
							489.94			22,369.84						
Anchor Forces																
AV1	0 - 59 Strand Anchors @ 45 degrees						0.00		39.375	0.00						
AH1	0 - 59 Strand Anchors @ 45 degrees							0.00	47.250	0.00						
AV2	0 - 59 Strand Anchors @ 45 degrees						0.00		30.750	0.00						
AH2	0 - 59 Strand Anchors @ 45 degrees							0.00	40.250	0.00						
AV3	0 - 59 Strand Anchors @ 45 degrees						0.00		22.125	0.00						
AH3	0 - 59 Strand Anchors @ 45 degrees							0.00	33.250	0.00						
MISC. VERTICAL																
W1	75.25 X	3 X	0.063 X	0.294 X	1		4.15		38.066	157.97						
CV	Net Vertical Crest Pressure & Moment						0.00			0.00						
R1	57.31 X	6.25 X	0.169 X	1 X	1		60.36		28.663	1,730.04						
UPLIFT																
U1	75.25 X	26.37 X	0.0625 X	1 X	-1		(124.02)		37.625	(4,666.08)						
U2	62.75 X	1.569 X	0.0625 X	1 X	-0.5		(3.08)		41.833	(128.69)						
U3	0 X	1.881 X	0.0625 X	1 X	-1		0.00		75.250	0.00						
U4	12.5 X	0.313 X	0.0625 X	1 X	-0.5		(0.12)		71.083	(8.68)						
U5	12.5 X	1.569 X	0.0625 X	1 X	-1		(1.23)		69.000	(84.57)						
U6	0 X	26.37 X	0.0625 X	1 X	1			0.00	0.000	0.00						
U7	0 X	1.569 X	0.0625 X	1 X	0.5			0.00	0.000	0.00						
U8	0 X	1.881 X	0.0625 X	1 X	1			0.00	0.000	0.00						
U9	0 X	0.313 X	0.0625 X	1 X	0.5			0.00	0.000	0.00						
U10	0 X	1.569 X	0.0625 X	1 X	1			0.00	0.000	0.00						
HYDROSTATIC																
H1	28.25 X	28.25 X	0.0625 X	1 X	-0.5			(24.94)	9.417	(234.85)						
H2	22.15 X	22.15 X	0.0625 X	1 X	0.5			15.33	7.383	113.20						
H3	0.0 X	0.0 X	0.0625 X	1 X	0.5			0.00	68.250	0.00						
MISC. HORIZONTAL																
E1	11 X	11 X	0.053 X	1 X	-0.5			(3.18)	10.917	(34.67)						
EQ	Seismic Momentum Load						(55.46)		32.190	(1,785.19)						
Sum V <u>426.01</u>						Sum H <u>(68.24)</u>										
Sum M <u> </u>						<u>17,428.31</u>										

$$M/V = 40.91 \text{ ft.}$$

$$e = M/V-B/2 = 3.29 \text{ ft.}$$

%Base in Compression = 100.0%

Min. Found. Pressure= 4.1782 ksf

Max. Found. Pressure= 7.1443 ksf

Bearing Capacity= 52.7 ksf

Sliding F.S. = 2.32

7.38



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 7 - Spillway
Apron Floatation and Sliding*

Computed By: SAW Date 01 Jun 2007

Checked By: ECV, RSR Date 01 Jun 2007

Apron Length = 102.00 Water Velocity Impacting Baffles = 35
Apron Phi = 9.5
Apron Cohesion = 0.25

Item	Width	Height	Length	Unit Wt.	Weight
Upstream Section					
Step 1	10.63 X	18.25 X	102.00 X	0.150 =	2966.77
Step 2	6.79 X	16.75 X	102.00 X	0.150 =	1740.53
Step 3	5.42 X	15.25 X	102.00 X	0.150 =	1263.84
Step 4	4.67 X	13.75 X	102.00 X	0.150 =	981.75
Step 5	5.17 X	12.25 X	102.00 X	0.150 =	968.36
Training Walls	26.50 X	10.00 X	20.62 X	0.150 =	819.49
Center Section					
Base Slab	49.58 X	12.25 X	102.00 X	0.150 =	9293.16
Baffle Step 1	1.75 X	1.25 X	86.25 X	0.150 =	28.30
Baffle Step 2	1.75 X	3.00 X	86.25 X	0.150 =	67.92
Baffle Step 3	5.50 X	7.50 X	86.25 X	0.150 =	533.67
Downstream Section					
Base Slab	42.50 X	12.25 X	102.00 X	0.150 =	7965.56
End Sill Step 1	2.00 X	2.00 X	102.00 X	0.150 =	61.20
End Sill Step 2	2.00 X	4.00 X	102.00 X	0.150 =	122.40
End Sill Step 3	2.00 X	6.00 X	102.00 X	0.150 =	183.60
Water Pressures					
90% or 60% TW	124.75 X	9.90 X	102.00 X	0.0625 =	7873.28
Uplift Pressures					
U1	124.75 X	23.25 X	102.00 X	-0.0625 =	-18490.29
U2	124.75 X	3.12 X	102.00 X	-0.0625 =	-1240.14
Apron Anchors					
	42 Anchors	X	282.74 kips/anchor	=	11875.08
				Sum V =	27014.49
				Lateral Load on Baffles =	9.42
				Floatation Factor of Safety =	3.28



US Army Corps
of Engineers
Huntington District

Dover Dam
Tuscarawas River
Dam Safety Assurance

COMPUTATION

*Monolith 7 - Spillway
Crest Pressures*

Computed By: SAW Date 01 Jun 2007
Checked By: ECV Date 01 Jun 2007

	Pool Elevation	Net Horizontal Force	Net Vertical Force	Net Moment
	916	-1.125	1.129	2.589
	917	-1.169	2.287	67.497
	918	-1.213	3.444	132.405
	919	-1.258	4.601	197.313
	920	-1.302	5.759	262.221
	921	-1.346	6.916	327.129
	922	-1.390	8.074	392.037
	923	-1.435	9.231	456.945
	924	-1.479	10.388	521.853
Calculated from Sap2000	924.5	-1.501	10.967	554.307
	925	-1.705	10.819	535.442
	926	-2.112	10.524	497.710
	927	-2.519	10.228	459.978
	928	-2.927	9.933	422.246
	929	-3.334	9.637	384.515
	930	-3.741	9.342	346.783
	931	-4.149	9.046	309.051
	932	-4.556	8.751	271.319
Calculated from Sap2000	933	-4.964	8.455	233.588
	934	-5.389	7.567	159.106
	935	-5.815	6.680	84.625
Calculated from Sap2000	935.89	-6.195	5.890	18.337
	936	-6.239	5.747	7.064
	937	-6.643	4.450	-95.418
	938	-7.048	3.154	-197.901
Calculated from Sap2000	938.61	-7.295	2.363	-260.415
	939	-7.458	1.648	-313.351
	940	-7.877	-0.185	-449.083
	941	-8.296	-2.018	-584.816
Calculated from Sap2000	941.50	-8.505	-2.935	-652.682



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 7 - Spillway
Seismic Analysis at Failure Plane 1 - OBE*

Computed By:	SAW	Date 01 Jun 2007
Checked By:	ECV, RSR	Date 01 Jun 2007

Key Concrete

Length	16.375 ft	Vertical Force in Key =	73.703
Length in Compression	16.375 ft	Concrete Phi =	38.000
U/S Found. Pressure	4.178 ksf		
D/S Pressure	4.824 ksf		

Key Reinforcement

Number of Bars Intersected	4.00		
Size of Bars	0.75 in		
Spacing of Bars	2.50 ft	Reinforcing Resisting Force =	9.90
Area of Bars per Ft	0.71 in ²	Equivalent Cohesion =	4.20
Shear Strength of Bars	14.00 ksi		

Shale Bedding Plane

Length	58.875 ft	Vertical Force on Shale =	352.308
Length in Compression	58.875 ft	Shale Phi =	9.50
U/S Found. Pressure	4.824 ksf	Shale Cohesion =	0.25
D/S Pressure	7.144 ksf		
		Wsk =	73.70
		Wss =	352.31
		Vs =	0.00
		Us =	0.00
		(HI-Hr)s =	68.24
		Ps =	-11.621

Apron

Wa =	264.67
Va =	193.61
Ua =	193.44
(HI-Hr)a =	9.42
Pa =	11.621

Passive Wedge

Wp	17.49 X	12.25 X	0.162 X	0.5 =	17.32
Vp	17.49 X	11.00 X	0.0625 X	1 =	12.03
	21.36 X	11.00 X	0.0625 X	1 =	14.68
	21.36 X	12.25 X	0.0625 X	0.5 =	8.18
				Up =	22.86
				(HI-Hr)p =	11.55
				Pp =	0.000

Sum Horizontal Forces = 0.00

Sliding Factor of Safety = 2.32



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 7 - Spillway
Seismic Analysis at Failure Plane 2 - OBE*

Computed By: SAW Date 01 Jun 2007
Checked By: ECV Date 01 Jun 2007

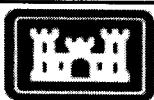
Structure Wedge	Width	Height	Length	Unit Wt.	Weight
Key Concrete		231.07 X	1.00 X	0.150 =	34.66
Shale		217.50 X	1.00 X	0.162 =	35.15
Sandstone		754.66 X	1.00 X	0.148 =	111.69
				Ws =	181.50
External Vertical Loads		Sum V from Failure Plane 1 Analysis - uplift		Vs =	554.45
Uplift 1	75.25 X	39.49 X	1.00 X	0.0625 =	185.73
Uplift 2	75.25 X	2.25 X	1.00 X	0.0625 =	5.29
				Us =	191.02
Hydrostatic Left	41.74 X	41.74 X	1.00 X	0.0625 =	54.44
Hydrostatic Right	36.37 X	36.37 X	1.00 X	-0.0625 =	-41.34
Silt Load		E1 from Failure Plane 1 Analysis		=	3.18
Horizontal Uplift on Plane 1		From Failure Plane 1 Analysis		=	0.00
				(HI-Hr)s =	16.28
				Ps =	-6.737
Passive Wedge (Rock)					
Shale		82.72 X	1.00 X	0.162 =	13.37
Sandstone		123.62 X	1.00 X	0.148 =	18.30
				Wr =	31.66
Vertical Load from Water	18.74 X	23.25 X	1.00 X	0.0625 =	27.23
Vertical Load from Apron				=	44.45
				Vr =	71.68
Uplift 1	22.88 X	25.62 X	1.00 X	0.0625 =	36.63
Uplift 2	22.88 X	13.87 X	1.00 X	0.0625 =	9.92
				Ur =	46.55
Hydrostatic Left	36.37 X	36.37 X	1.00 X	0.0625 =	41.34
Hydrostatic Left	23.25 X	23.25 X	1.00 X	-0.0625 =	-16.89
				(HI-Hr)r =	24.45
				Pr =	0.000
Passive Wedge (Apron)					
Concrete	17.49 X	12.25 X	1.00 X	0.1500 =	16.07
				Wa =	16.07
Vertical Load from Water	17.49 X	11.00 X	1.00 X	0.0625 =	12.03
				Va =	12.03
Uplift 1	21.36 X	11.00 X	1.00 X	0.0625 =	14.68
Uplift 2	21.36 X	14.62 X	1.00 X	0.0625 =	9.76
				Ua =	24.44
Hydrostatic Left	23.25 X	23.25 X	1.00 X	0.0625 =	16.89
Hydrostatic Left	11.00 X	11.00 X	1.00 X	-0.0625 =	-3.78
				(HI-Hr)a =	13.11
				Pa =	6.737
				Ps + Pp =	0.00
				Sliding Factor of Safety =	55.36

INPUTS	Var. Name	Notes
Head Water Elevation	870	HW
Tail Water Elevation	865	TW
Failure Plane 1 Elevation	841.75	BaseEl
Failure Plane 1 Angle	0	BaseS
Toe Elevation 1	841.75	ToeEL
Failure Plane 2 Elevation	828.26	BaseEl2
Failure Plane 2 Angle	0.28	BaseS2
Toe Elevation 1	828.63	ToeEL2
Drain Efficiency	0%	DrainEff
Top Elevation	916	TopEl
Base Width	75.25	BaseW
Length of Monolith	34	MonLength
Analysis Length	1	Length
Backfill Elevation U/S	860	BFUS
Backfill Elevation D/S	854	BFDS
Backfill Ko	1	Ko
Backfill Gamma moist	0.11	gamma_m in kcf
Backfill Gamma Sat.	0.115	gamma_sat in kcf
Failure Plane 1 Cohesion	0.25	BaseC in psi
Failure Plane 1 phi	9.50	BasePhi
Failure Plane 2 Cohesion	2.50	BaseC2 in psi
Failure Plane 2 phi	29.00	BasePhi2
Crossbed Cohesion	0.00	XBedC
Crossbed Phi	19.00	XBedPhi
Crossbed Failure Angle	35.00	XBedAng
Gamma Rock	0.169	gamma_r
Top Rock US	849.00	RockUS Must be greater than or equal to Base Elevation
Top Rock DS	854.00	RockDS Must be greater than or equal to Base Elevation
Bearing Capacity	52.7	BearCap in ksf
Seismic Coefficient	0.15	ac MCE
Row 1 Anchors		
# of Anchors per Mono.	0	
# of Strands per Anchor	59	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	45.0	Angle down from horizontal in degrees
Elevation of Anchors	889.0	AnchEl Elevation Anchors are Installed
Distance from Toe	39.4	AnchLoc Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	0.00	V_Anch Calculated in Kips per Length
Horizontal Anchor Force	0.00	H_Anch Calculated in Kips per Length
Row 2 Anchors		
# of Anchors per Mono.	0	
# of Strands per Anchor	59	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	45.0	Angle down from horizontal in degrees
Elevation of Anchors	882.0	AnchEl2 Elevation Anchors are Installed
Distance from Toe	30.8	AnchLoc2 Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	0.00	V_Anch2 Calculated in Kips per Length
Horizontal Anchor Force	0.00	H_Anch2 Calculated in Kips per Length
Row 3 Anchors		
# of Anchors per Mono.	0	
# of Strands per Anchor	59	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	45.0	Angle down from horizontal in degrees
Elevation of Anchors	875.0	AnchEl3 Elevation Anchors are Installed
Distance from Toe	22.1	AnchLoc3 Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	0.00	V_Anch3 Calculated in Kips per Length
Horizontal Anchor Force	0.00	H_Anch3 Calculated in Kips per Length

Dover DSA Monolith 7

899
864

Inputs



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 7 - Spillway
Deducts*

Computed By: SAW Date 01 Jun 2007

Checked By: ECV Date 01 Jun 2007

Item	Width	Height	Length	Volume	Arm	Moment
Gallery - Rect. Portion	6.00 X	15.00 X	34.00 X	3060.00 X	62.75	= 192015.0
Gallery - Arch Portion	6.00 X	3.00 X	34.00 X	480.66 X	62.75	= 30161.6
Sluice Gate Mach. Recess	6.00 X	18.58 X	18.00 X	2007.00 X	56.75	= 113897.3
Sluice Gate Mach. Recess	8.25 X	2.25 X	18.00 X	334.13 X	56.75	= 18961.6
Sluice Gate Shaft Ext.	4.00 X	8.40 X	2.00 X	211.01 X	56.75	= 11974.8
Pipe Conduit	2.50 X	2.50 X	16.00 X	78.54 X	54.65	= 4291.9
Air Inlet	4.00 X	4.00 X	34.00 X	427.26 X	48.75	= 20828.8
Bulkhead Slot	2.00 X	10.25 X	14.00 X	287.00 X	63	= 18081.0
Sluice Gate Slot	2.00 X	9.25 X	13.00 X	240.50 X	58.25	= 14009.1
Sluice	75.25 X	10.00 X	10.00 X	7525.00 X	38.066	= 286446.7
				<u>14651.10</u>		<u>710667.7</u>
Equivalent Square Deduct	20.76 X	20.76 X	34 X	14651.10 X	48.51	= 710667.7
Item	Width	Height	Length	Volume	Arm	Moment
Rock - Rect. Portion	55.75 X	6.25 X	34.00 X	11846.88 X	27.88	= 330231.6
Rock - Tri. Portion	3.13 X	6.25 X	34.00 X	332.03 X	56.79	= 18856.6
				<u>12178.91</u>		<u>349088.2</u>
Equivalent Square Rock	57.31 X	6.25 X	34.00 X	12178.91 X	28.66	= 349088.2



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 7 - Spillway
Seismic Analysis - MCE*

Computed By:	SAW	Date	01 Jun 2007
Checked By:	ECV, RSR	Date	01 Jun 2007

	Width	X	Height	X	Unit Wt.	X	Length	Vertical	Horizontal	Arm	Moments					
CONCRETE																
C1								554.58		45.990	25,505.13					
C2	20.76 X	20.76 X	0.15 X	1 X	-1			(64.64)		48.506	(3,135.30)					
Concrete Subtotal =																
Anchor Forces																
AV1	0 - 59 Strand Anchors @ 45 degrees							0.00		39.375	0.00					
AH1	0 - 59 Strand Anchors @ 45 degrees								0.00	47.250	0.00					
AV2	0 - 59 Strand Anchors @ 45 degrees							0.00		30.750	0.00					
AH2	0 - 59 Strand Anchors @ 45 degrees								0.00	40.250	0.00					
AV3	0 - 59 Strand Anchors @ 45 degrees							0.00		22.125	0.00					
AH3	0 - 59 Strand Anchors @ 45 degrees								0.00	33.250	0.00					
MISC. VERTICAL																
W1	75.25 X	3 X	0.063 X	0.294 X	1			4.15		38.066	157.97					
CV	Net Vertical Crest Pressure & Moment							0.00			0.00					
R1	57.31 X	6.25 X	0.169 X	1 X	1			60.36		28.663	1,730.04					
UPLIFT																
U1	75.25 X	26.37 X	0.0625 X	1 X	-1			(124.02)		37.625	(4,666.08)					
U2	62.75 X	1.569 X	0.0625 X	1 X	-0.5			(3.08)		41.833	(128.69)					
U3	0 X	1.881 X	0.0625 X	1 X	-1			0.00		75.250	0.00					
U4	12.5 X	0.313 X	0.0625 X	1 X	-0.5			(0.12)		71.083	(8.68)					
U5	12.5 X	1.569 X	0.0625 X	1 X	-1			(1.23)		69.000	(84.57)					
U6	0 X	26.37 X	0.0625 X	1 X	1				0.00	0.000	0.00					
U7	0 X	1.569 X	0.0625 X	1 X	0.5				0.00	0.000	0.00					
U8	0 X	1.881 X	0.0625 X	1 X	1				0.00	0.000	0.00					
U9	0 X	0.313 X	0.0625 X	1 X	0.5				0.00	0.000	0.00					
U10	0 X	1.569 X	0.0625 X	1 X	1				0.00	0.000	0.00					
HYDROSTATIC																
H1	28.25 X	28.25 X	0.0625 X	1 X	-0.5				(24.94)	9.417	(234.85)					
H2	22.15 X	22.15 X	0.0625 X	1 X	0.5				15.33	7.383	113.20					
H3	0.0 X	0.0 X	0.0625 X	1 X	0.5				0.00	68.250	0.00					
MISC. HORIZONTAL																
E1	11 X	11 X	0.053 X	1 X	-0.5				(3.18)	10.917	(34.67)					
EQ	Seismic Momentum Load								(83.19)	32.190	(2,677.79)					
								Sum V	<u>426.01</u>	Sum H	<u>(95.97)</u>					
								Sum M		<u>16,535.72</u>						

M/V = 38.82 ft.
e = M/V-B/2= 1.19 ft.
%Base in Compression = 100.0%

Min. Found. Pressure= 5.124 ksf
Max. Found. Pressure= 6.1986 ksf
Bearing Capacity= 52.7 ksf

Sliding F.S. = 1.77
Bearing F.S. = 8.50



US Army Corps
of Engineers
Huntington District

Dover Dam
Tuscarawas River
Dam Safety Assurance

COMPUTATION

*Monolith 7 - Spillway
Apron Floatation and Sliding*

Computed By: SAW Date 01 Jun 2007

Checked By: ECV, RSR Date 01 Jun 2007

Apron Length = 102.00 Water Velocity Impacting Baffles = 35
Apron Phi = 9.5
Apron Cohesion = 0.25

Item	Width	Height	Length	Unit Wt.	Weight
Upstream Section					
Step 1	10.63 X	18.25 X	102.00 X	0.150 =	2966.77
Step 2	6.79 X	16.75 X	102.00 X	0.150 =	1740.53
Step 3	5.42 X	15.25 X	102.00 X	0.150 =	1263.84
Step 4	4.67 X	13.75 X	102.00 X	0.150 =	981.75
Step 5	5.17 X	12.25 X	102.00 X	0.150 =	968.36
Training Walls	26.50 X	10.00 X	20.62 X	0.150 =	819.49
Center Section					
Base Slab	49.58 X	12.25 X	102.00 X	0.150 =	9293.16
Baffle Step 1	1.75 X	1.25 X	86.25 X	0.150 =	28.30
Baffle Step 2	1.75 X	3.00 X	86.25 X	0.150 =	67.92
Baffle Step 3	5.50 X	7.50 X	86.25 X	0.150 =	533.67
Downstream Section					
Base Slab	42.50 X	12.25 X	102.00 X	0.150 =	7965.56
End Sill Step 1	2.00 X	2.00 X	102.00 X	0.150 =	61.20
End Sill Step 2	2.00 X	4.00 X	102.00 X	0.150 =	122.40
End Sill Step 3	2.00 X	6.00 X	102.00 X	0.150 =	183.60
Water Pressures					
90% or 60% TW	124.75 X	9.90 X	102.00 X	0.0625 =	7873.28
Uplift Pressures					
U1	124.75 X	23.25 X	102.00 X	-0.0625 =	-18490.29
U2	124.75 X	3.12 X	102.00 X	-0.0625 =	-1240.14
Apron Anchors					
	42 Anchors	X	282.74 kips/anchor	=	11875.08
				Sum V =	27014.49
				Lateral Load on Baffles =	9.42
				Floatation Factor of Safety =	3.28



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 7 - Spillway
Crest Pressures*

Computed By: SAW Date 01 Jun 2007
Checked By: ECV Date 01 Jun 2007

	Pool Elevation	Net Horizontal Force	Net Vertical Force	Net Moment
	916	-1.125	1.129	2.589
	917	-1.169	2.287	67.497
	918	-1.213	3.444	132.405
	919	-1.258	4.601	197.313
	920	-1.302	5.759	262.221
	921	-1.346	6.916	327.129
	922	-1.390	8.074	392.037
	923	-1.435	9.231	456.945
	924	-1.479	10.388	521.853
Calculated from Sap2000	924.5	-1.501	10.967	554.307
	925	-1.705	10.819	535.442
	926	-2.112	10.524	497.710
	927	-2.519	10.228	459.978
	928	-2.927	9.933	422.246
	929	-3.334	9.637	384.515
	930	-3.741	9.342	346.783
	931	-4.149	9.046	309.051
	932	-4.556	8.751	271.319
Calculated from Sap2000	933	-4.964	8.455	233.588
	934	-5.389	7.567	159.106
	935	-5.815	6.680	84.625
Calculated from Sap2000	935.89	-6.195	5.890	18.337
	936	-6.239	5.747	7.064
	937	-6.643	4.450	-95.418
	938	-7.048	3.154	-197.901
Calculated from Sap2000	938.61	-7.295	2.363	-260.415
	939	-7.458	1.648	-313.351
	940	-7.877	-0.185	-449.083
	941	-8.296	-2.018	-584.816
Calculated from Sap2000	941.50	-8.505	-2.935	-652.682



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 7 - Spillway
Seismic Analysis at Failure Plane 1 - MCE*

Computed By:	SAW	Date	01 Jun 2007
Checked By:	ECV, RSR	Date	01 Jun 2007

Key Concrete

Length	16.375 ft	Vertical Force in Key =	85.820
Length in Compression	16.375 ft	Concrete Phi =	38.000
U/S Found. Pressure	5.124 ksf		
D/S Pressure	5.358 ksf		

Key Reinforcement

Number of Bars Intersected	4.00		
Size of Bars	0.75 in		
Spacing of Bars	2.50 ft	Reinforcing Resisting Force =	9.90
Area of Bars per Ft	0.71 in ²	Equivalent Cohesion =	4.20
Shear Strength of Bars	14.00 ksi		

Shale Bedding Plane

Length	58.875 ft	Vertical Force on Shale =	340.191
Length in Compression	58.875 ft	Shale Phi =	9.50
U/S Found. Pressure	5.358 ksf	Shale Cohesion =	0.25
D/S Pressure	6.199 ksf		
		Wsk =	85.82
		Wss =	340.19
		Vs =	0.00
		Us =	0.00
		(HI-Hr)s =	95.97
		Ps =	-18.173

Apron

Wa =	264.67
Va =	193.61
Ua =	193.44
(HI-Hr)a =	9.42
Pa =	18.173

Passive Wedge

Wp	17.49 X	12.25 X	0.162 X	0.5 =	17.32
Vp	17.49 X	11.00 X	0.0625 X	1 =	12.03
	21.36 X	11.00 X	0.0625 X	1 =	14.68
	21.36 X	12.25 X	0.0625 X	0.5 =	8.18
				Up =	22.86
				(HI-Hr)p =	11.55
				Pp =	0.000

Sum Horizontal Forces = 0.00

Sliding Factor of Safety = 1.77



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

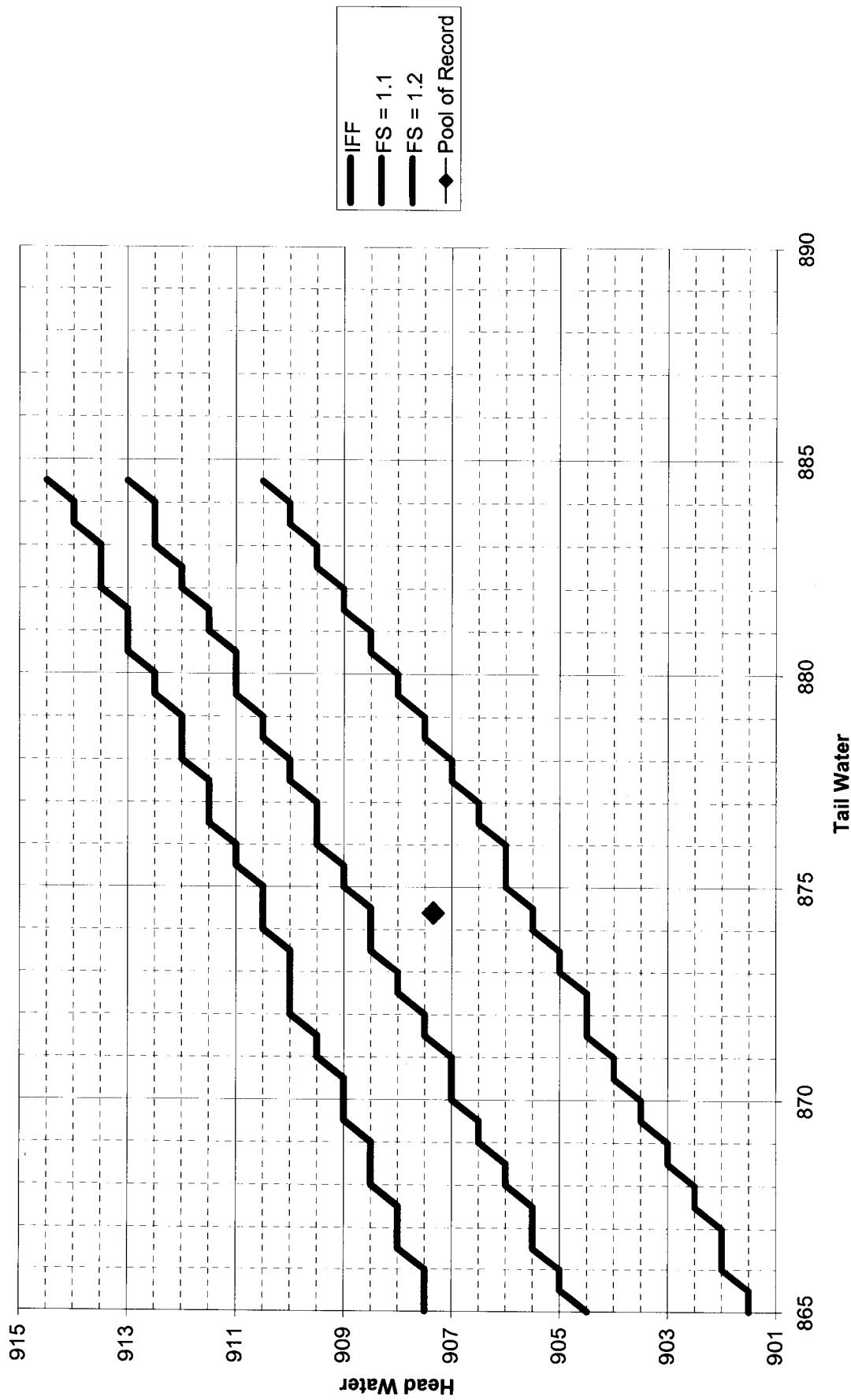
COMPUTATION

*Monolith 7 - Spillway
Seismic Analysis at Failure Plane 2 - MCE*

Computed By:	SAW	Date	01 Jun 2007
Checked By:	ECV	Date	01 Jun 2007

Structure Wedge	Width	Height	Length	Unit Wt.	Weight
Key Concrete		231.07 X	1.00 X	0.150 =	34.66
Shale		217.50 X	1.00 X	0.162 =	35.15
Sandstone		754.66 X	1.00 X	0.148 =	111.69
				Ws =	181.50
External Vertical Loads		Sum V from Failure Plane 1 Analysis - uplift		Vs =	554.45
Uplift 1	75.25 X	39.49 X	1.00 X	0.0625 =	185.73
Uplift 2	75.25 X	2.25 X	1.00 X	0.0625 =	5.29
				Us =	191.02
Hydrostatic Left	41.74 X	41.74 X	1.00 X	0.0625 =	54.44
Hydrostatic Right	36.37 X	36.37 X	1.00 X	-0.0625 =	-41.34
Silt Load		E1 from Failure Plane 1 Analysis		=	3.18
Horizontal Uplift on Plane 1		From Failure Plane 1 Analysis		=	0.00
				(Hi-Hr)s =	16.28
				Ps =	-6.737
Passive Wedge (Rock)					
Shale		82.72 X	1.00 X	0.162 =	13.37
Sandstone		123.62 X	1.00 X	0.148 =	18.30
				Wr =	31.66
Vertical Load from Water	18.74 X	23.25 X	1.00 X	0.0625 =	27.23
Vertical Load from Apron				=	44.45
				Vr =	71.68
Uplift 1	22.88 X	25.62 X	1.00 X	0.0625 =	36.63
Uplift 2	22.88 X	13.87 X	1.00 X	0.0625 =	9.92
				Ur =	46.55
Hydrostatic Left	36.37 X	36.37 X	1.00 X	0.0625 =	41.34
Hydrostatic Left	23.25 X	23.25 X	1.00 X	-0.0625 =	-16.89
				(Hi-Hr)r =	24.45
				Pr =	0.000
Passive Wedge (Apron)					
Concrete	17.49 X	12.25 X	1.00 X	0.1500 =	16.07
				Wa =	16.07
Vertical Load from Water	17.49 X	11.00 X	1.00 X	0.0625 =	12.03
				Va =	12.03
Uplift 1	21.36 X	11.00 X	1.00 X	0.0625 =	14.68
Uplift 2	21.36 X	14.62 X	1.00 X	0.0625 =	9.76
				Ua =	24.44
Hydrostatic Left	23.25 X	23.25 X	1.00 X	0.0625 =	16.89
Hydrostatic Left	11.00 X	11.00 X	1.00 X	-0.0625 =	-3.78
				(Hi-Hr)a =	13.11
				Pa =	6.737
				Ps + Pp =	0.00
				Sliding Factor of Safety =	55.36

Dover IFF (Pool Combinations Yielding FS 1.0)



Dover DSA Monolith 7 IFF

INPUTS	Var. Name	Notes
Head Water Elevation	915	HW
Tail Water Elevation	884.5	TW
Failure Plane 1 Elevation	841.75	BaseEl
Failure Plane 1 Angle	0	BaseS
Toe Elevation 1	841.75	ToeEL
Failure Plane 2 Elevation	828.26	BaseEl2
Failure Plane 2 Angle	0.28	BaseS2
Toe Elevation 1	828.63	ToeEL2
Drain Efficiency	10%	DrainEff
Top Elevation	916	TopEl
Base Width	75.25	BaseW
Length of Monolith	34	MonLength
Analysis Length	1	Length
Backfill Elevation U/S	860	BFUS
Backfill Elevation D/S	854	BFDS
Backfill Ko	1	Ko
Backfill Gamma moist	0.11	gamma_m in kcf
Backfill Gamma Sat.	0.115	gamma_sat in kcf
Failure Plane 1 Cohesion	0.25	BaseC in psi
Failure Plane 1 phi	9.50	BasePhi
Failure Plane 2 Cohesion	2.50	BaseC2 in psi
Failure Plane 2 phi	29.00	BasePhi2
Crossbed Cohesion	0.00	XBedC
Crossbed Phi	19.00	XBedPhi
Crossbed Failure Angle	35.00	XBedAng
Gamma Rock	0.169	gamma_r
Top Rock US	849.00	RockUS Must be greater than or equal to Base Elevation
Top Rock DS	854.00	RockDS Must be greater than or equal to Base Elevation
Bearing Capacity	52.7	BearCap in ksf
Row 1 Anchors		
# of Anchors per Mono.	0	
# of Strands per Anchor	0	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	45.0	Angle down from horizontal in degrees
Elevation of Anchors	889.0	AnchEl Elevation Anchors are Installed
Distance from Toe	39.4	AnchLoc Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	0.00	V_Anch Calculated in Kips per Length
Horizontal Anchor Force	0.00	H_Anch Calculated in Kips per Length
Row 2 Anchors		
# of Anchors per Mono.	0	
# of Strands per Anchor	0	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	45.0	Angle down from horizontal in degrees
Elevation of Anchors	882.0	AnchEl2 Elevation Anchors are Installed
Distance from Toe	30.8	AnchLoc2 Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	0.00	V_Anch2 Calculated in Kips per Length
Horizontal Anchor Force	0.00	H_Anch2 Calculated in Kips per Length
Row 3 Anchors		
# of Anchors per Mono.	0	
# of Strands per Anchor	0	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	90.0	Angle down from horizontal in degrees
Elevation of Anchors	849.0	AnchEl3 Elevation Anchors are Installed
Distance from Toe	0.0	AnchLoc3 Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	0.00	V_Anch3 Calculated in Kips per Length
Horizontal Anchor Force	0.00	H_Anch3 Calculated in Kips per Length



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 7 - Spillway
Deducts*

Computed By: SAW Date 01 Jun 2007

Checked By: ECV Date 01 Jun 2007

Item	Width	Height	Length	Volume	Arm	Moment
Gallery - Rect. Portion	6.00 X	15.00 X	34.00 X	3060.00 X	62.75 =	192015.0
Gallery - Arch Portion	6.00 X	3.00 X	34.00 X	480.66 X	62.75 =	30161.6
Sluice Gate Mach. Recess	6.00 X	18.58 X	18.00 X	2007.00 X	56.75 =	113897.3
Sluice Gate Mach. Recess	8.25 X	2.25 X	18.00 X	334.13 X	56.75 =	18961.6
Sluice Gate Shaft Ext.	4.00 X	8.40 X	2.00 X	211.01 X	56.75 =	11974.8
Pipe Conduit	2.50 X	2.50 X	16.00 X	78.54 X	54.65 =	4291.9
Air Inlet	4.00 X	4.00 X	34.00 X	427.26 X	48.75 =	20828.8
Bulkhead Slot	2.00 X	10.25 X	14.00 X	287.00 X	63 =	18081.0
Sluice Gate Slot	2.00 X	9.25 X	13.00 X	240.50 X	58.25 =	14009.1
Sluice	75.25 X	10.00 X	10.00 X	7525.00 X	38.066 =	286446.7
				<u>14651.10</u>		<u>710667.7</u>
Equivalent Square Deduct	20.76 X	20.76 X	34 X	14651.10 X	48.51 =	710667.7
Item	Width	Height	Length	Volume	Arm	Moment
Rock - Rect. Portion	55.75 X	6.25 X	34.00 X	11846.88 X	27.88 =	330231.6
Rock - Tri. Portion	3.13 X	6.25 X	34.00 X	332.03 X	56.79 =	18856.6
				<u>12178.91</u>		<u>349088.2</u>
Equivalent Square Rock	57.31 X	6.25 X	34.00 X	12178.91 X	28.66 =	349088.2



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

**Monolith 7 - Spillway
IFF Analysis Stability Sheet**

							Computed By:	SAW	Date	01 Jun 2007
							Checked By:	ECV	Date	01 Jun 2007
							Vertical	Horizontal	Arm	Moments
	Width	X	Height	X	Unit Wt.	X	Length			
CONCRETE										
C1	Dam Concrete from Microstation						554.58		45.990	25,505.13
C2	20.76	X	20.76	X	0.15	X	1	-1	(64.64)	(3,135.30)
	Concrete Subtotal =						489.94			22,369.84
Anchor Forces										
AV1	0 - 0 Strand Anchors @ 45 degrees						0.00		39.375	0.00
AH1	0 - 0 Strand Anchors @ 45 degrees							0.00	47.250	0.00
AV2	0 - 0 Strand Anchors @ 45 degrees						0.00		30.750	0.00
AH2	0 - 0 Strand Anchors @ 45 degrees							0.00	40.250	0.00
AV3	0 - 0 Strand Anchors @ 90 degrees						0.00		0.000	0.00
AH3	0 - 0 Strand Anchors @ 90 degrees							0.00	7.250	0.00
MISC. VERTICAL										
W1	75.25	X	10	X	0.063	X	0.294	X	1	13.83
CV	Net Vertical Crest Pressure & Moment						0.00		38.066	526.56
R1	57.31	X	6.25	X	0.169	X	1	X	1	60.36
UPLIFT										
U1	75.25	X	59.87	X	0.0625	X	1	X	-1	(281.59)
U2	62.75	X	8.612	X	0.0625	X	1	X	-0.5	(16.89)
U3	0	X	13.38	X	0.0625	X	1	X	-1	0.00
U4	12.5	X	4.766	X	0.0625	X	1	X	-0.5	(1.86)
U5	12.5	X	8.612	X	0.0625	X	1	X	-1	(6.73)
U6	0	X	59.87	X	0.0625	X	1	X	1	
U7	0	X	8.612	X	0.0625	X	1	X	0.5	
U8	0	X	13.38	X	0.0625	X	1	X	1	
U9	0	X	4.766	X	0.0625	X	1	X	0.5	
U10	0	X	8.612	X	0.0625	X	1	X	1	
HYDROSTATIC										
H1	73.25	X	73.25	X	0.0625	X	1	X	-0.5	
H2	39.70	X	39.70	X	0.0625	X	1	X	0.5	
H3	0.0	X	0.0	X	0.0625	X	1	X	0.5	
MISC. HORIZONTAL										
E1	11	X	11	X	0.053	X	1	X	-0.5	
CH	Net Horizontal Crest Pressure							(3.18)	10.917	(34.67)
							Sum V	<u>257.07</u>	Sum H	<u>(121.60)</u>
									Sum M	<u>9,251.77</u>

$$M/V = 35.99 \text{ ft.}$$

$$e = M/V/2 = -1.64 \text{ ft.}$$

%Base in Compression = 100.0%

$$\text{Sliding F.S.} = 0.973$$

$$13.647$$

$$\text{Min. Found. Pressure} = 2.9707 \text{ ksf}$$

$$\text{Max. Found. Pressure} = 3.8617 \text{ ksf}$$

$$\text{Bearing Capacity} = 52.7 \text{ ksf}$$



**US Army Corps
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Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 7 - Spillway
Apron Floatation and Sliding*

Computed By: SAW Date 01 Jun 2007
Checked By: ECV Date 01 Jun 2007

Apron Length = 102.00 Water Velocity Impacting Baffles = 35
Apron Phi = 9.5
Apron Cohesion = 0.25

Item	Width	Height	Length	Unit Wt.	Weight
Upstream Section					
Step 1	10.63 X	18.25 X	102.00 X	0.150 =	2966.77
Step 2	6.79 X	16.75 X	102.00 X	0.150 =	1740.53
Step 3	5.42 X	15.25 X	102.00 X	0.150 =	1263.84
Step 4	4.67 X	13.75 X	102.00 X	0.150 =	981.75
Step 5	5.17 X	12.25 X	102.00 X	0.150 =	968.36
Training Walls	26.50 X	10.00 X	20.62 X	0.150 =	819.49

Center Section					
Base Slab	49.58 X	12.25 X	102.00 X	0.150 =	9293.16
Baffle Step 1	1.75 X	1.25 X	86.25 X	0.150 =	28.30
Baffle Step 2	1.75 X	3.00 X	86.25 X	0.150 =	67.92
Baffle Step 3	5.50 X	7.50 X	86.25 X	0.150 =	533.67

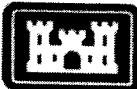
Downstream Section					
Base Slab	42.50 X	12.25 X	102.00 X	0.150 =	7965.56
End Sill Step 1	2.00 X	2.00 X	102.00 X	0.150 =	61.20
End Sill Step 2	2.00 X	4.00 X	102.00 X	0.150 =	122.40
End Sill Step 3	2.00 X	6.00 X	102.00 X	0.150 =	183.60

Water Pressures					
90% or 60% TW	124.75 X	27.45 X	102.00 X	0.0625 =	21830.47
Uplift Pressures					
U1	124.75 X	42.75 X	102.00 X	-0.0625 =	-33998.27
U2	124.75 X	17.12 X	102.00 X	-0.0625 =	-6808.38

Sum V = 8020.37

Lateral Load on Baffles = 9.42

Floatation Factor of Safety = 1.42



US Army Corps
of Engineers
Huntington District

Dover Dam
Tuscarawas River
Dam Safety Assurance

COMPUTATION

*Monolith 7 - Spillway
Crest Pressures*

Computed By: SAW Date 01 Jun 2007
Checked By: ECV Date 01 Jun 2007

	Pool Elevation	Net Horizontal Force	Net Vertical Force	Net Moment
	916	-1.125	1.129	2.589
	917	-1.169	2.287	67.497
	918	-1.213	3.444	132.405
	919	-1.258	4.601	197.313
	920	-1.302	5.759	262.221
	921	-1.346	6.916	327.129
	922	-1.390	8.074	392.037
	923	-1.435	9.231	456.945
	924	-1.479	10.388	521.853
Calculated from Sap2000	924.5	-1.501	10.967	554.307
	925	-1.705	10.819	535.442
	926	-2.112	10.524	497.710
	927	-2.519	10.228	459.978
	928	-2.927	9.933	422.246
	929	-3.334	9.637	384.515
	930	-3.741	9.342	346.783
	931	-4.149	9.046	309.051
	932	-4.556	8.751	271.319
Calculated from Sap2000	933	-4.964	8.455	233.588
	934	-5.389	7.567	159.106
	935	-5.815	6.680	84.625
Calculated from Sap2000	935.89	-6.195	5.890	18.337
	936	-6.239	5.747	7.064
	937	-6.643	4.450	-95.418
	938	-7.048	3.154	-197.901
Calculated from Sap2000	938.61	-7.295	2.363	-260.415
	939	-7.458	1.648	-313.351
	940	-7.877	-0.185	-449.083
	941	-8.296	-2.018	-584.816
Calculated from Sap2000	941.50	-8.505	-2.935	-652.682



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 7 - Spillway
IFF Analysis at Failure Plane 1*

Computed By:	SAW	Date 01 Jun 2007
Checked By:	ECV	Date 01 Jun 2007

Key Concrete

Length	16.375 ft	Vertical Force in Key =	50.233
Length in Compression	16.375 ft	Concrete Phi =	38.000
U/S Found. Pressure	2.971 ksf		
D/S Pressure	3.165 ksf		

Key Reinforcement

Number of Bars Intersected	4.00		
Size of Bars	0.75 in		
Spacing of Bars	2.50 ft	Reinforcing Resisting Force =	9.90
Area of Bars per Ft	0.71 in ²	Equivalent Cohesion =	4.20
Shear Strength of Bars	14.00 ksi		

Shale Bedding Plane

Length	58.875 ft	Vertical Force on Shale =	206.837
Length in Compression	58.875 ft	Shale Phi =	9.50
U/S Found. Pressure	3.165 ksf	Shale Cohesion =	0.25
D/S Pressure	3.862 ksf		
		Wsk =	50.23
		Wss =	206.84
		Vs =	0.00
		Us =	0.00
		(HI-Hr)s =	121.60
		Ps =	-33.430

Apron

Wa =	264.67
Va =	214.02
Ua =	400.07
(HI-Hr)a =	9.42
Pa =	8.713

Passive Wedge

Wp	17.49 X	12.25 X	0.162 X	0.5 =	17.32
	8.57 X	6.00 X	0.028 X	1 =	1.41
Vp	17.49 X	30.50 X	0.0625 X	1 =	34.76
	21.36 X	30.50 X	0.0625 X	1 =	40.71
	21.36 X	12.25 X	0.0625 X	0.5 =	8.18
				Up =	48.89
				(HI-Hr)p =	20.18
				Pp =	24.717

Sum Horizontal Forces = 0.00

Sliding Factor of Safety = 0.97



**US Army Corps
of Engineers**
Huntington District

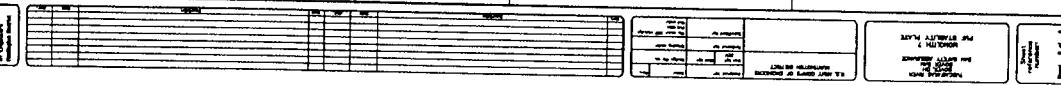
**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 7 - Spillway
IFF Analysis at Failure Plane 2*

Computed By:	SAW	Date 01 Jun 2007
Checked By:	ECV	Date 01 Jun 2007

Structure Wedge	Width	Height	Length	Unit Wt.	Weight
Key Concrete		231.07 X	1.00 X	0.150 =	34.66
Shale		217.50 X	1.00 X	0.162 =	35.15
Sandstone		754.66 X	1.00 X	0.148 =	111.69
				Ws =	181.50
External Vertical Loads		Sum V from Failure Plane 1 Analysis - uplift			
Uplift 1	75.25 X	72.99 X	1.00 X	0.0625 =	343.30
Uplift 2	75.25 X	13.75 X	1.00 X	0.0625 =	32.32
				Us =	375.63
Hydrostatic Left	86.74 X	86.74 X	1.00 X	0.0625 =	235.12
Hydrostatic Right	55.87 X	55.87 X	1.00 X	-0.0625 =	-97.55
Silt Load		E1 from Failure Plane 1 Analysis			
Horizontal Uplift on Plane 1		From Failure Plane 1 Analysis			
				=	3.18
				=	0.00
				(HI-Hr)s =	140.74
				Ps =	-64.306
Passive Wedge (Rock)					
Shale		82.72 X	1.00 X	0.162 =	13.37
Sandstone		123.62 X	1.00 X	0.148 =	18.30
				Wr =	31.66
Vertical Load from Water	18.74 X	42.75 X	1.00 X	0.0625 =	50.07
Vertical Load from Apron				=	44.45
				Vr =	94.52
Uplift 1	22.88 X	55.75 X	1.00 X	0.0625 =	79.72
Uplift 2	22.88 X	17.24 X	1.00 X	0.0625 =	12.33
				Ur =	92.05
Hydrostatic Left	55.87 X	55.87 X	1.00 X	0.0625 =	97.55
Hydrostatic Left	42.75 X	42.75 X	1.00 X	-0.0625 =	-57.11
				(HI-Hr)r =	40.44
				Pr =	56.744
Passive Wedge (Apron)					
Concrete	17.49 X	12.25 X	1.00 X	0.1500 =	16.07
				Wa =	16.07
Vertical Load from Water	17.49 X	30.50 X	1.00 X	0.0625 =	33.35
				Va =	33.35
Uplift 1	21.36 X	30.50 X	1.00 X	0.0625 =	40.71
Uplift 2	21.36 X	25.25 X	1.00 X	0.0625 =	16.86
				Ua =	57.57
Hydrostatic Left	42.75 X	42.75 X	1.00 X	0.0625 =	57.11
Hydrostatic Left	30.50 X	30.50 X	1.00 X	-0.0625 =	-29.07
				(HI-Hr)a =	28.04
				Pa =	7.563
				Ps + Pp =	0.00
				Sliding Factor of Safety =	3.19



EQUATION PARAMETERS

1. ON DRAIN EFFECTIVENESS
2. ALONG WEAK PLANE: $\phi_m = 9.5^\circ$, $C = 0.26$ PSI
REDUCED DUE TO STRAIN COMPATIBILITY
3. WEAK PLANE ANGLE 1 = 0 DEGREES
4. ALONG WEAK PLANE 2: $\phi_m = 26^\circ$, $C = 2.5$ PSI
5. ALONG CROSSED PLANE: $\phi_m = 19^\circ$, $C = 0$ PSI
6. WEAK PLANE ANGLE 2 = 0.28
7. ROCK WEDGE FAILURE ANGLE = 35.0
8. ALLOWABLE BEARING CAPACITY = 336 PSI = 52.7 KSF

F.S. SLIDING = 1.10

BASE IN COMPRESSION = 100%

NOTES: 1. THE ANALYSIS WAS PERFORMED FOR A 100% LOAD.

2. THE ANALYSIS WAS PERFORMED FOR A 100% LOAD.

3. THE ANALYSIS WAS PERFORMED FOR A 100% LOAD.

4. THE ANALYSIS WAS PERFORMED FOR A 100% LOAD.

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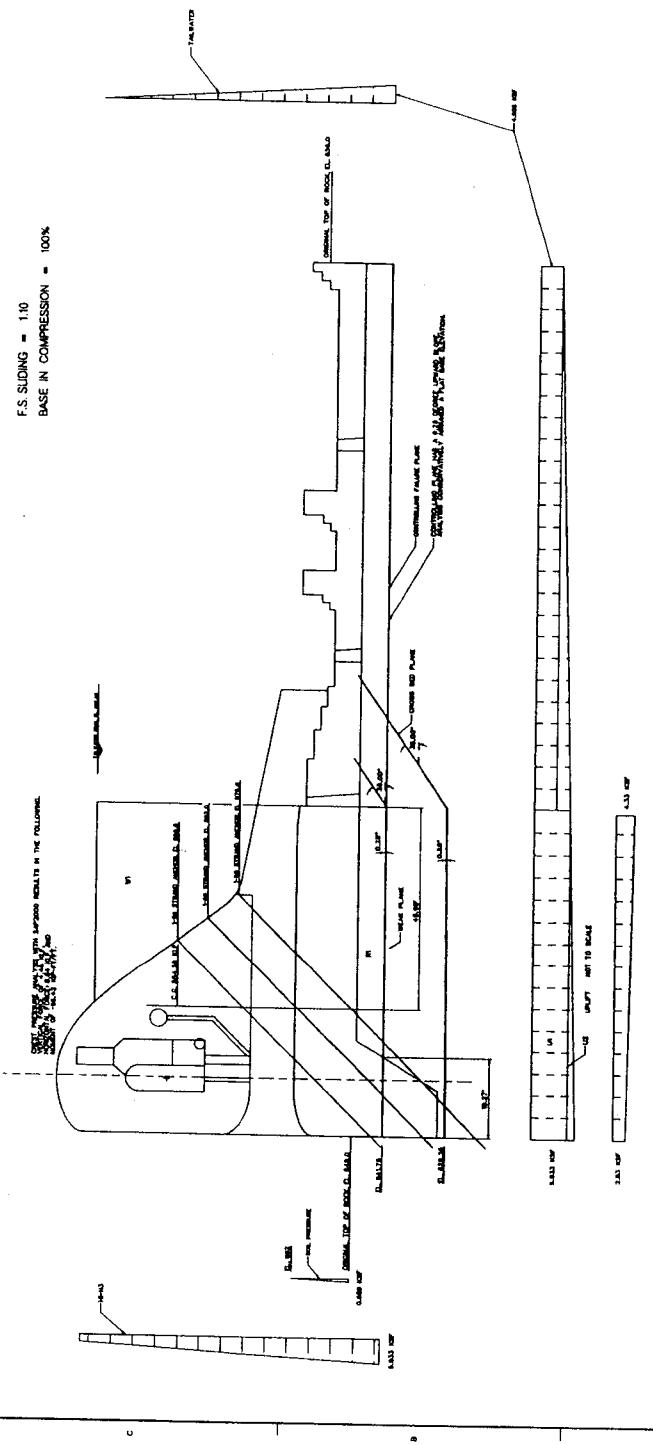
96. THE ANALYSIS WAS PERFORMED FOR A 100% LOAD.

97. THE ANALYSIS WAS PERFORMED FOR A 100% LOAD.

98. THE ANALYSIS WAS PERFORMED FOR A 100% LOAD.

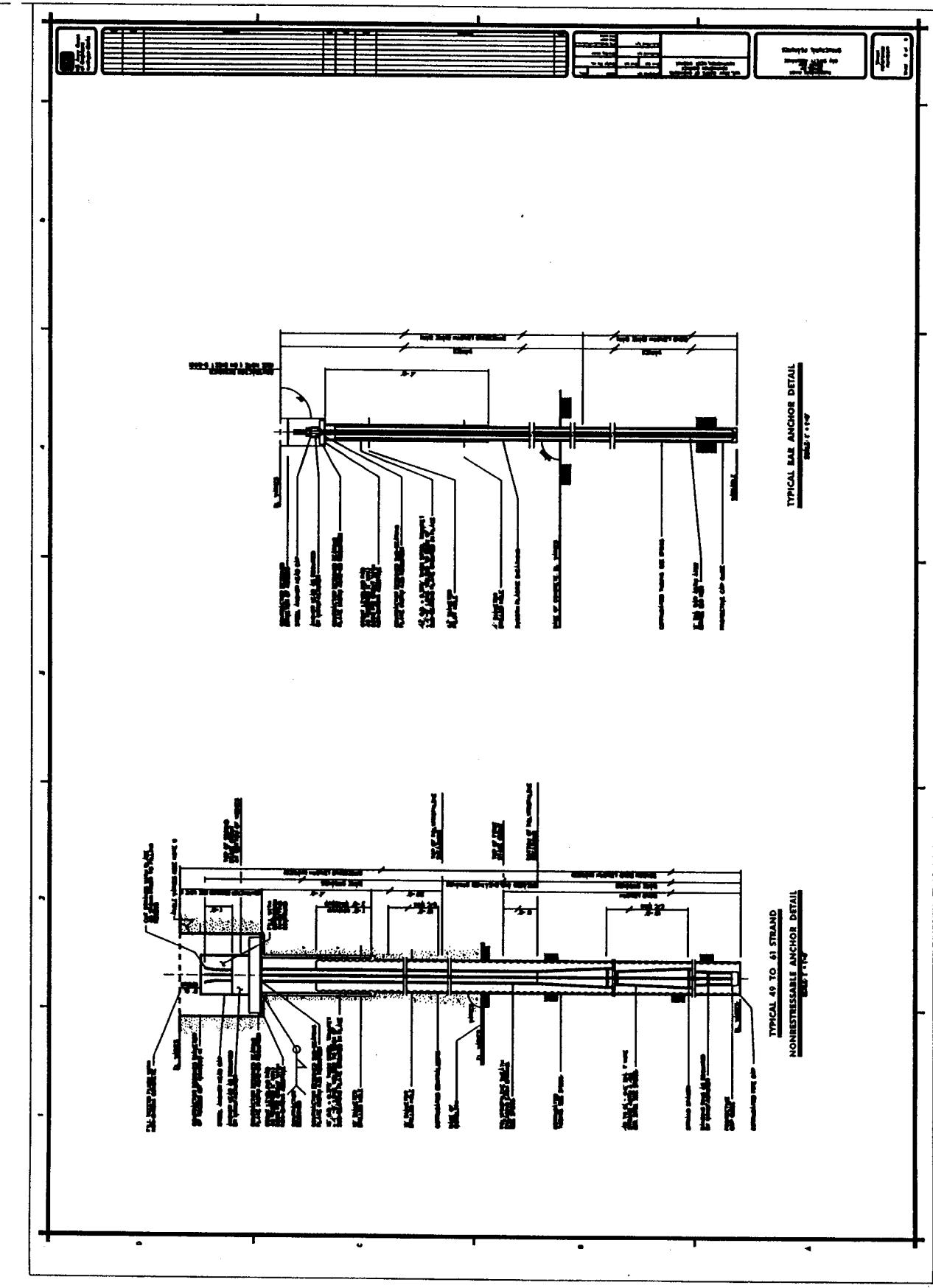
99. THE ANALYSIS WAS PERFORMED FOR A 100% LOAD.

100. THE ANALYSIS WAS PERFORMED FOR A 100% LOAD.



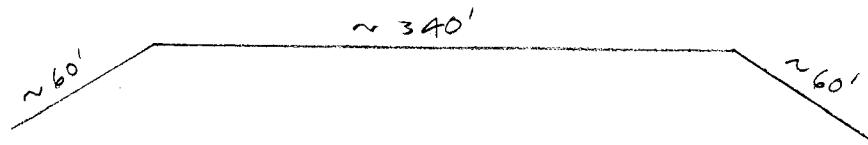
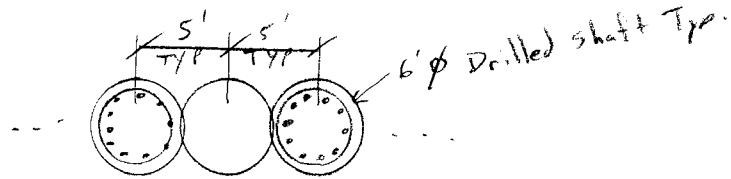
MONOUTH 7
STA 3+70
100 ft.
LOAD CASE 3: PMF

FOUNDATION PROFILE



Dover DSAPER | Spillway Cut-Off | SAW 11/13/06
 ✓ ECV

Assume Secant-Type Wall



91 shafts - 46 reinforced, 45 un-reinforced

Base Elevation = 830

Average height = 34 ft

$$\text{Concrete} = 34 \times \pi 3^2 / 27 \times 91 = \underline{\underline{3,240 \text{ cu yds}}}$$

Assume 12 - #9's vertical
 #3 spiral, $s=3''$

$$12 \times 3.40 \times 34 \times 46 = 64,000 \text{ lb}$$

$$(\pi \times s) \times \frac{34}{0.25} \times 0.376 \times 46 = \underline{\underline{37,000 \text{ lb}}}$$

$$\text{Reinforcing} = \underline{\underline{101,000 \text{ lb}}}$$

Monolith 17

Case 1: Spillway Crest

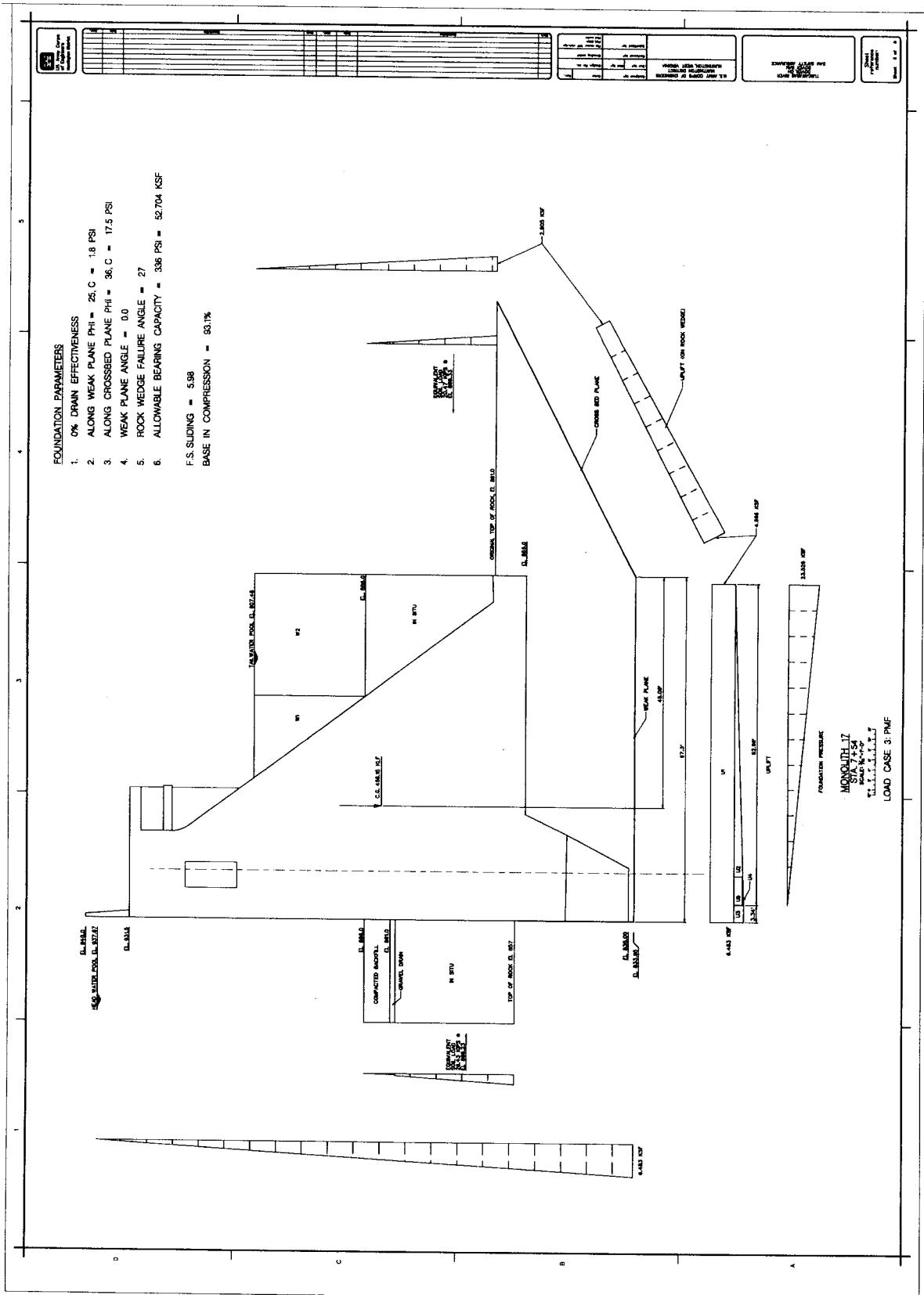
Head water	EL 916.0
Tail water	EL 868.0
Base in Compression	100 %
F.S. Sliding	5.0
F.S. Bearing	3.6

Case 2: 300 year event

Head water	EL 918.3
Tail water	EL 881.4
Base in Compression	100 %
F.S. Sliding	6.6
F.S. Bearing	4.8

Case 3: PMF

Head water	EL 937.67
Tail water	EL 907.48
Base in Compression	93.1 %
F.S. Sliding	6.0
F.S. Bearing	3.3



DOVER DSA MONOLITH 17

INPUTS		Var. Name	Notes
Head Water Elevation	916	HW	
Tail Water Elevation	868	TW	
Failure Plane Elevation	833.95	BaseEl	
Failure Plane Slope	0	BaseS	
Toe Elevation	833.95	ToeEL	
Drain Efficiency	0%	DrainEff	
Top Elevation	931.5	TopEl	
Base Width	67.3	BaseW	
Length of Monolith	36	MonLength	
Analysis Length	1	Length	
Backfill Elevation U/S	886	BFUS	
Backfill Elevation D/S	886	BFDS	
Backfill Ko	Ko		
Backfill Gamma moist	gamma_m	in kcf	
Backfill Gamma Sat.	gamma_sat	in kcf	
Failure Plane Cohesior	1.80	BaseC	in psi
Failure Plane phi	25.00	BasePhi	
Crossbed Cohesion	17.50	XBedC	
Crossbed Phi	36.00	XBedPhi	
Crossbed Failure Angle	27.00	XBedAng	
Gamma Rock	0.162	gamma_r	
Top Rock US	857.00	RockUS	Must be greater than or equal to Base Elevation
Top Rock DS	861.00	RockDS	Must be greater than or equal to Base Elevation
Bearing Capacity	52.7	BearCap	
Row 1 Anchors			
# of Anchors per Mono	0		
# of Strands per Anchor	0		270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	90.0		Angle down from horizontal in degrees
Elevation of Anchors	849.0	AnchEl	Elevation Anchors are Installed
Distance from Toe	0.0	AnchLoc	Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	0.00	V_Anch	Calculated in Kips per Length
Horizontal Anchor Force	0.00	H_Anch	Calculated in Kips per Length
Row 2 Anchors			
# of Anchors per Mono	0		
# of Strands per Anchor	0		270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	90.0		Angle down from horizontal in degrees
Elevation of Anchors	849.0	AnchEl2	Elevation Anchors are Installed
Distance from Toe	0.0	AnchLoc2	Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	0.00	V_Anch2	Calculated in Kips per Length
Horizontal Anchor Force	0.00	H_Anch2	Calculated in Kips per Length
Row 3 Anchors			
# of Anchors per Mono	0		
# of Strands per Anchor	0		270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	90.0		Angle down from horizontal in degrees
Elevation of Anchors	849.0	AnchEl3	Elevation Anchors are Installed
Distance from Toe	0.0	AnchLoc3	Distance from Toe of Monolith Anchors are Installed
Vertical Anchor Force	0.00	V_Anch3	Calculated in Kips per Length
Horizontal Anchor Force	0.00	H_Anch3	Calculated in Kips per Length



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 5 - Right Abutment
Deducts*

Computed By: SAW Date 22 Nov 2006

Checked By: ECV Date 22 Nov 2006

Item	Width	Height	Length	Volume	Arm	Moment
Stairway	5.00 X	10.00 X	36.00 X	1800.00 X	58.80 =	105840.0
	X	X	X	0.00 X	=	0.0
	X	X	X	0.00 X	=	0.0
	X	X	X	0.00 X	=	0.0
	X	X	X	0.00 X	=	0.0
	X	X	X	0.00 X	=	0.0
	X	X	X	0.00 X	=	0.0
	X	X	X	0.00 X	=	0.0
				<u>1800.00</u>		<u>105840.0</u>
Equivalent Square Deduct	7.07 X	7.07 X	36 X	1800.00 X	58.80 =	105840.0



**US Army Corps
of Engineers**
Huntington District

Dover Dam Tuscarawas River Dam Safety Assurance

COMPUTATION

*Monolith 17 - Right Abutment
Load Case 1 at Base Elevation 833.95*

							Computed By:	ECV	Date	22 Nov 2006
							Checked By:	SAW	Date	22 Nov 2006
Width	X	Height	X	Unit Wt.	X	Length	Vertical	Horizontal	Arm	Moments
CONCRETE										
C1		Dam Concrete from Microstation					456.15		45.086	20,566.10
C2		Key Concrete from Microstation					24.01		60.370	1,449.69
C3	7.07 X	7.07 X	0.145 X	1 X	-1		(7.25)		58.800	(426.30)
		Concrete Subtotal =					472.92			21,589.49
Anchor Forces										
AV1	0 - 0 Strand Anchors @ 90 degrees						0.00		0.000	0.00
AH1	0 - 0 Strand Anchors @ 90 degrees							0.00	15.050	0.00
AV2	0 - 0 Strand Anchors @ 90 degrees						0.00		0.000	0.00
AH2	0 - 0 Strand Anchors @ 90 degrees							0.00	15.050	0.00
AV3	0 - 0 Strand Anchors @ 90 degrees						0.00		0.000	0.00
AH3	0 - 0 Strand Anchors @ 90 degrees							0.00	15.050	0.00
MISC. VERTICAL										
W1	13.5 X	18 X	1.000 X	1 X	0		0.00		359.031	0.00
W2	354.53 X	18 X	0.000 X	1 X	0		0.00		177.266	0.00
S1	304.09 X	1 X	0.125 X	1 X	1		38.01		8.896	0.00
S2	50.444 X	1 X	0.128 X	1 X	1		6.46		3.978	338.14
R1	67.3 X	16.166 X	0.162 X	1 X	1		175.82		33.650	25.69
										5,916.26
UPLIFT										
U1	67.3 X	34.05 X	0.0625 X	1 X	-1		(143.22)		33.650	(4,819.45)
U2	58.3 X	41.581 X	0.0625 X	1 X	-0.5		(75.76)		38.867	(2,944.36)
U3	0 X	48 X	0.0625 X	1 X	-1		0.00		67.300	0.00
U4	9 X	6.419 X	0.0625 X	1 X	-0.5		(1.81)		62.800	(113.38)
U5	9 X	41.581 X	0.0625 X	1 X	-1		(23.39)		64.300	(1,503.93)
U6	0 X	34.05 X	0.0625 X	1 X	-1			0.00	0.000	0.00
U7	0 X	41.581 X	0.0625 X	1 X	-0.5			0.00	0.000	0.00
U8	0 X	48 X	0.0625 X	1 X	-1			0.00	0.000	0.00
U9	0 X	6.419 X	0.0625 X	1 X	-0.5			0.00	0.000	0.00
U10	0 X	41.581 X	0.0625 X	1 X	-1			0.00	0.000	0.00
HYDROSTATIC										
H1	82.05 X	82.05 X	0.0625 X	1 X	-0.5			(210.38)	27.350	(5,753.93)
H2	34.1 X	34.1 X	0.0625 X	1 X	0.5			36.23	11.350	411.23
MISC. HORIZONTAL										
E1	26.43 X	1 X	1.000 X	1 X	-1			(26.43)	32.280	(853.16)
E2	20.469 X	1 X	1.000 X	1 X	1			20.47	35.380	724.19
E3	X	X	0.000 X	1 X	-1				27.050	
E4	X	X	0.000 X	1 X	-1				27.050	

$$M/V = -28.99 \text{ ft.}$$

$$e = M/V - B/2 = -4.66 \text{ ft.}$$

%Base in Compression = 100.0%

Sliding F.S.= 4.954
Bearing F.S.= 3.615

Max. Found. Pressure= 14.581 ksf
Bearing Capacity= 52.704 ksf

Wedge Analysis

FS =	4.954
	0.950
Inputs	
Crossbed c =	17.500
Crossbed ϕ =	36.000
rad =	0.628
Unit Weight of Rock =	0.162
c at Base =	1.800
ϕ at Base =	25.000
rad =	0.436
Top Rock U/S	857.00
Top Rock D/S	861.00
Upper Pool	916.00
Lower Pool	868.00

Active Wedge	
αa =	-49.172
rad =	-0.858
Wa =	
Va =	
Ua =	
La =	
Pa =	0.000
Sum =	0.000

Passive Wedge	
αp =	27.000
rad =	0.471
Wp =	18.771
Vp =	87.882
Up =	-16.681
Lp =	30.359
Hl-Hr =	-7.573
Pp =	104.907

Structure Wedge	
αs =	0.000
rad =	0.000
Ws =	472.916
Vs =	44.468
Us =	-244.173
Ls =	67.300
Hl-Hr =	180.111
Ps =	-104.907

Analysis Results

PROGRAM CSOILP - CALCULATION OF AT-REST PRESSURES BY COMBINATION
OF "GRAVITY-TURN-ON" AND THEORY OF ELASTICITY COMPONENTS
DATE: 18-AUGUST-06 TIME: 9:39:24

* RESULTS *

I.--HEADING
'M17 DS load case 1

II.--EQUIVALENT SURCHARGE LOAD DUE TO IRREGULAR SURFACE ABOVE 886.0 (FT)
NONE

III.--PRESSURES ON WALL BELOW EL. 886.0 (FT)

ELEVATION (FT)	GRAVITY- TURN-ON PRESSURE (PSF)	ELASTICITY COMPONENT PRESSURE (PSF)	COMBINED GTO & ELASTICITY PRESSURE (PSF)
886.00	0.00	0.00	0.00
885.00	65.50	0.00	65.50
884.00	131.00	0.00	131.00
883.00	196.50	0.00	196.50
882.00	262.00	0.00	262.00
881.00	327.50	0.00	327.50
880.00	393.00	0.00	393.00
879.00	458.50	0.00	458.50
878.00	524.00	0.00	524.00
877.00	589.50	0.00	589.50
876.00	655.00	0.00	655.00
875.00	720.50	0.00	720.50
874.00	786.00	0.00	786.00
873.00	851.50	0.00	851.50
872.00	917.00	0.00	917.00
871.00	982.50	0.00	982.50
870.00	1048.00	0.00	1048.00
869.00	1113.50	0.00	1113.50
868.00	1179.00	0.00	1179.00
867.00	1244.50	0.00	1244.50
866.00	1310.00	0.00	1310.00
865.00	1375.50	0.00	1375.50
864.00	1441.00	0.00	1441.00
863.00	1506.50	0.00	1506.50
862.00	1572.00	0.00	1572.00
861.00+	1637.50	0.00	1637.50

Resultant Force for Combined Pressures = 20468.75 lbs
Resultant Location at elev. = 869.33 feet

Input Data

PROGRAM CSOILP - CALCULATION OF AT-REST PRESSURES BY COMBINATION
OF "GRAVITY-TURN-ON" AND THEORY OF ELASTICITY COMPONENTS
DATE: 18-AUGUST-06 TIME: 9:39:28

* INPUT DATA *

I.--HEADING
'M17 DS load case 1

II.--SURFACE POINTS

DIST. FROM WALL (FT)	ELEVATION (FT)
0.0	886.0

III.--SOIL LAYER DATA

<WEIGHT SAT.	(PCF)> MST.	INTERNAL (DEG)	ELASTICITY COEFF.	BOTTOM ELEV. (FT)
128.0	125.0	0.0	0.00	861.0

IV.--WATER DATA

UNIT WEIGHT:	62.5 (PCF)
ELEVATION:	916.0 (FT)

DOVER DSA MONOLITH 17

INPUTS	Var. Name	Notes
Head Water Elevation	918.3	HW
Tail Water Elevation	881.4	TW
Failure Plane Elevation	833.95	BaseEl
Failure Plane Slope	0	BaseS
Toe Elevation	833.95	ToeEL
Drain Efficiency	0%	DrainEff
Top Elevation	931.5	TopEl
Base Width	67.3	BaseW
Length of Monolith	36	MonLength
Analysis Length	1	Length
Backfill Elevation U/S	886	BFUS
Backfill Elevation D/S	886	BFDS
Backfill Ko	Ko	
Backfill Gamma moist	gamma_m	in kcf
Backfill Gamma Sat.	gamma_sat	in kcf
Failure Plane Cohesior	1.80	BaseC
Failure Plane phi	25.00	BasePhi
Crossbed Cohesion	17.50	XBedC
Crossbed Phi	36.00	XBedPhi
Crossbed Failure Angle	27.00	XBedAng
Gamma Rock	0.162	gamma_r
Top Rock US	857.00	RockUS
Top Rock DS	861.00	RockDS
Bearing Capacity	52.7	BearCap
Row 1 Anchors		
# of Anchors per Mono	0	
# of Strands per Anchor	0	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	90.0	Angle down from horizontal in degrees
Elevation of Anchors	849.0	AnchEl
Distance from Toe	0.0	AnchLoc
Vertical Anchor Force	0.00	V_Anch
Horizontal Anchor Force	0.00	H_Anch
Row 2 Anchors		
# of Anchors per Mono	0	
# of Strands per Anchor	0	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	90.0	Angle down from horizontal in degrees
Elevation of Anchors	849.0	AnchEl2
Distance from Toe	0.0	AnchLoc2
Vertical Anchor Force	0.00	V_Anch2
Horizontal Anchor Force	0.00	H_Anch2
Row 3 Anchors		
# of Anchors per Mono	0	
# of Strands per Anchor	0	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	90.0	Angle down from horizontal in degrees
Elevation of Anchors	849.0	AnchEl3
Distance from Toe	0.0	AnchLoc3
Vertical Anchor Force	0.00	V_Anch3
Horizontal Anchor Force	0.00	H_Anch3



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 5 - Right Abutment
Deducts*

Computed By: SAW Date 22 Nov 2006

Checked By: ECV Date 22 Nov 2006

Item	Width	Height	Length	Volume	Arm	Moment
Stairway	5.00 X	10.00 X	36.00 X	1800.00 X	58.80 =	105840.0
	X	X	X	0.00 X	=	0.0
	X	X	X	0.00 X	=	0.0
	X	X	X	0.00 X	=	0.0
	X	X	X	0.00 X	=	0.0
	X	X	X	0.00 X	=	0.0
	X	X	X	0.00 X	=	0.0
	X	X	X	0.00 X	=	0.0
	X	X	X	0.00 X	=	0.0
				<u>1800.00</u>		<u>105840.0</u>
Equivalent Square Deduct	7.07 X	7.07 X	36 X	1800.00 X	58.80 =	105840.0



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 17 - Right Abutment
Load Case 2 at Base Elevation 833.95*

Computed By:	<u>ECV</u>	Date	22 Nov 2006
Checked By:	<u>SAW</u>	Date	22 Nov 2006

	Width	X	Height	X	Unit Wt.	X	Length	Vertical	Horizontal	Arm	Moments
CONCRETE											
C1								456.15		45.086	20,566.10
C2								24.01		60.370	1,449.69
C3	7.07 X	7.07 X	0.145 X	1 X	-1			(7.25)		58.800	(426.30)
								472.92			21,589.49
Anchor Forces											
AV1	0 - 0 Strand Anchors @ 90 degrees							0.00		0.000	0.00
AH1	0 - 0 Strand Anchors @ 90 degrees								0.00	15.050	0.00
AV2	0 - 0 Strand Anchors @ 90 degrees							0.00		0.000	0.00
AH2	0 - 0 Strand Anchors @ 90 degrees								0.00	15.050	0.00
AV3	0 - 0 Strand Anchors @ 90 degrees							0.00		0.000	0.00
AH3	0 - 0 Strand Anchors @ 90 degrees								0.00	15.050	0.00
MISC. VERTICAL											
W1	3.45 X	4.6 X	1.000 X	1 X	0			0.00		355.680	0.00
W2	354.53 X	4.6 X	0.000 X	1 X	0			0.00		177.265	0.00
S1	100.83 X	1 X	0.125 X	1 X	1			12.60		10.980	138.39
S2	253.7 X	1 X	0.128 X	1 X	1			32.47		7.090	230.24
R1	67.3 X	16.166 X	0.162 X	1 X	1			175.82		33.650	5,916.26
UPLIFT											
U1	67.3 X	47.45 X	0.0625 X	1 X	-1		(199.59)			33.650	(6,716.09)
U2	58.3 X	31.965 X	0.0625 X	1 X	-0.5		(58.24)			38.867	(2,263.48)
U3	0 X	36.9 X	0.0625 X	1 X	-1		0.00			67.300	0.00
U4	9 X	4.9346 X	0.0625 X	1 X	-0.5		(1.39)			62.800	(87.16)
U5	9 X	31.965 X	0.0625 X	1 X	-1		(17.98)			64.300	(1,156.15)
U6	0 X	47.45 X	0.0625 X	1 X	-1			0.00		0.000	0.00
U7	0 X	31.965 X	0.0625 X	1 X	-0.5			0.00		0.000	0.00
U8	0 X	36.9 X	0.0625 X	1 X	-1			0.00		0.000	0.00
U9	0 X	4.9346 X	0.0625 X	1 X	-0.5			0.00		0.000	0.00
U10	0 X	31.965 X	0.0625 X	1 X	-1			0.00		0.000	0.00
HYDROSTATIC											
H1	84.35 X	84.35 X	0.0625 X	1 X	-0.5				(222.34)	28.117	(6,251.50)
H2	47.4 X	47.4 X	0.0625 X	1 X	0.5				70.36	15.817	1,112.85
MISC. HORIZONTAL											
E1	26.43 X	1 X	1.000 X	1 X	-1				(26.43)	32.280	(853.16)
E2	26.682 X	1 X	1.000 X	1 X	1				26.68	36.090	962.95
E3	X	X	0.000 X	1 X	-1					27.050	
E4	X	X	0.000 X	1 X	-1					27.050	
	Sum V	<u>416.62</u>	Sum H	<u>(151.73)</u>	Sum M	<u>12,622.66</u>					

$$M/V = 30.30 \text{ ft.}$$

$$e = M/V \cdot B/2 = -3.35 \text{ ft.}$$

%Base in Compression = 100.0%

Sliding F.S.= 6.646
Bearing F.S.= 4.789

Max. Found. Pressure= 11.006 ksf
Bearing Capacity= 52.704 ksf

Wedge Analysis

FS =	6.646	Active Wedge	Passive Wedge	Structure Wedge
	0.950			
Inputs				
Crossbed c =	17.500	$\alpha_a = -48.119$	$\alpha_p = 27.000$	$\alpha_s = 0.000$
Crossbed ϕ =	36.000	$rad = -0.840$	$rad = 0.471$	$rad = 0.000$
$rad =$	0.628	$Wa =$	$W_p = 18.771$	$W_s = 472.916$
Unit Weight of Rock =	0.162	$V_a =$	$V_p = 87.882$	$V_s = 45.077$
c at Base =	1.800	$U_a =$	$U_p = -16.681$	$U_s = -277.192$
ϕ at Base =	25.000	$L_a =$	$L_p = 30.359$	$L_s = 67.300$
$rad =$	0.436	$P_a = 0.000$	$H_i - H_r = -7.573$	$H_i - H_r = 151.730$
Top Rock U/S	857.00	$Sum = 0.000$	$P_p = 93.315$	$P_s = -93.315$
Top Rock D/S	861.00			
Upper Pool	918.30			
Lower Pool	881.40			

Analysis Results

PROGRAM CSOILP - CALCULATION OF AT-REST PRESSURES BY COMBINATION
OF "GRAVITY-TURN-ON" AND THEORY OF ELASTICITY COMPONENTS
DATE: 17-AUGUST-06 TIME: 14:49:56

* RESULTS *

I.--HEADING
'M17 DS 300 year event

II.--EQUIVALENT SURCHARGE LOAD DUE TO IRREGULAR SURFACE ABOVE 886.0 (FT)
NONE

III.--PRESSURES ON WALL BELOW EL. 886.0 (FT)

ELEVATION (FT)	GRAVITY- TURN-ON PRESSURE (PSF)	ELASTICITY COMPONENT PRESSURE (PSF)	COMBINED GTO & ELASTICITY PRESSURE (PSF)
886.00	0.00	0.00	0.00
885.00	125.00	0.00	125.00
884.00	250.00	0.00	250.00
883.00	375.00	0.00	375.00
882.00	500.00	0.00	500.00
881.40	575.00	0.00	575.00
881.00	601.20	0.00	601.20
880.00	666.70	0.00	666.70
879.00	732.20	0.00	732.20
878.00	797.70	0.00	797.70
877.00	863.20	0.00	863.20
876.00	928.70	0.00	928.70
875.00	994.20	0.00	994.20
874.00	1059.70	0.00	1059.70
873.00	1125.20	0.00	1125.20
872.00	1190.70	0.00	1190.70
871.00	1256.20	0.00	1256.20
870.00	1321.70	0.00	1321.70
869.00	1387.20	0.00	1387.20
868.00	1452.70	0.00	1452.70
867.00	1518.20	0.00	1518.20
866.00	1583.70	0.00	1583.70
865.00	1649.20	0.00	1649.20
864.00	1714.70	0.00	1714.70
863.00	1780.20	0.00	1780.20
862.00	1845.70	0.00	1845.70
861.00+	1911.20	0.00	1911.20

Resultant Force for Combined Pressures = 26681.74 lbs
Resultant Location at elev. = 870.04 feet

Input Data

PROGRAM CSOILP - CALCULATION OF AT-REST PRESSURES BY COMBINATION
OF "GRAVITY-TURN-ON" AND THEORY OF ELASTICITY COMPONENTS
DATE: 17-AUGUST-06 TIME: 14:50:03

* INPUT DATA *

I.--HEADING
'M17 DS 300 year event

II.--SURFACE POINTS

DIST. FROM WALL (FT)	ELEVATION (FT)
0.0	886.0

III.--SOIL LAYER DATA

<WEIGHT (PCF)>	SAT.	MST.	INTERNAL FRICTION (DEG)	ELASTICITY COEFF.	BOTTOM ELEV. (FT)
128.0	125.0		0.0	0.00	861.0

IV.--WATER DATA

UNIT WEIGHT:	62.5 (PCF)
ELEVATION:	881.4 (FT)

DOVER DSA MONOLITH 17

INPUTS	Var. Name	Notes
Head Water Elevation	937.67	HW
Tail Water Elevation	907.48	TW
Failure Plane Elevation	833.95	BaseEI
Failure Plane Slope	0	BaseS
Toe Elevation	833.95	ToeEL
Drain Efficiency	0%	DrainEff
Top Elevation	931.5	TopEI
Base Width	67.3	BaseW
Length of Monolith	36	MonLength
Analysis Length	1	Length
Backfill Elevation U/S	886	BFUS
Backfill Elevation D/S	886	BFDS
Backfill Ko	Ko	
Backfill Gamma moist	gamma_m	in kcf
Backfill Gamma Sat.	gamma_sat	in kcf
Failure Plane Cohesion	1.80	BaseC
Failure Plane phi	25.00	BasePhi
Crossbed Cohesion	17.50	XBedC
Crossbed Phi	36.00	XBedPhi
Crossbed Failure Angle	27.00	XBedAng
Gamma Rock	0.162	gamma_r
Top Rock US	857.00	RockUS
Top Rock DS	861.00	RockDS
Bearing Capacity	52.7	BearCap
Row 1 Anchors		
# of Anchors per Mono	0	
# of Strands per Anchor	0	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	90.0	Angle down from horizontal in degrees
Elevation of Anchors	849.0	AnchEI
Distance from Toe	0.0	AnchLoc
Vertical Anchor Force	0.00	V_Anch
Horizontal Anchor Force	0.00	H_Anch
Row 2 Anchors		
# of Anchors per Mono	0	
# of Strands per Anchor	0	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	90.0	Angle down from horizontal in degrees
Elevation of Anchors	849.0	AnchEI2
Distance from Toe	0.0	AnchLoc2
Vertical Anchor Force	0.00	V_Anch2
Horizontal Anchor Force	0.00	H_Anch2
Row 3 Anchors		
# of Anchors per Mono	0	
# of Strands per Anchor	0	270 ksi, 0.6" dia. Strands, 35.17 Kips per Strand Design Load
Angle of Anchors	90.0	Angle down from horizontal in degrees
Elevation of Anchors	849.0	AnchEI3
Distance from Toe	0.0	AnchLoc3
Vertical Anchor Force	0.00	V_Anch3
Horizontal Anchor Force	0.00	H_Anch3



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 5 - Right Abutment
Deducts*

Computed By: SAW Date 22 Nov 2006

Checked By: ECV Date 22 Nov 2006

Item	Width	Height	Length	Volume	Arm	Moment
Stairway	5.00 X	10.00 X	36.00 X	1800.00 X	58.80 =	105840.0
	X	X	X	0.00 X	=	0.0
	X	X	X	0.00 X	=	0.0
	X	X	X	0.00 X	=	0.0
	X	X	X	0.00 X	=	0.0
	X	X	X	0.00 X	=	0.0
	X	X	X	0.00 X	=	0.0
	X	X	X	0.00 X	=	0.0
				1800.00		105840.0
Equivalent Square Deduct	7.07 X	7.07 X	36 X	1800.00 X	58.80 =	105840.0



**US Army Corps
of Engineers**
Huntington District

**Dover Dam
Tuscarawas River
Dam Safety Assurance**

COMPUTATION

*Monolith 17 - Right Abutment
Load Case 3: PMF Analysis at Base Elevation 833.95*

Computed By:	ECV	Date	22 Nov 2006
Checked By:	SAW	Date	22 Nov 2006

Width	X	Height	X	Unit Wt.	X	Length	Vertical	Horizontal	Arm	Moments		
CONCRETE												
C1		Dam Concrete from Microstation					456.15		45.086	20,566.10		
C2		Key Concrete from Microstation					24.01		60.370	1,449.69		
C3	7.07 X	7.07 X	0.145 X	1 X	-1		(7.25)		58.800	(426.30)		
		Concrete Subtotal =					472.92			21,589.49		
Anchor Forces												
AV1	0 - 0 Strand Anchors @ 90 degrees						0.00		0.000	0.00		
AH1	0 - 0 Strand Anchors @ 90 degrees							0.00	15.050	0.00		
AV2	0 - 0 Strand Anchors @ 90 degrees						0.00		0.000	0.00		
AH2	0 - 0 Strand Anchors @ 90 degrees							0.00	15.050	0.00		
AV3	0 - 0 Strand Anchors @ 90 degrees						0.00		0.000	0.00		
AH3	0 - 0 Strand Anchors @ 90 degrees							0.00	15.050	0.00		
MISC. VERTICAL												
W1	16.11 X	21.48 X	0.0625 X	1 X	0.5		10.81		29.010	313.71		
W2	23.64 X	21.48 X	0.0625 X	1 X	1		31.74		11.820	375.13		
S1	34.86 X	46.48 X	0.000 X	1 X	0		0.00		366.151	0.00		
S2	354.53 X	1 X	0.128 X	1 X	1		45.38		8.197	371.96		
R1	67.3 X	16.166 X	0.162 X	1 X	1		175.82		33.650	5,916.26		
UPLIFT												
U1	67.3 X	73.53 X	0.0625 X	1 X	-1		(309.29)		33.650	(10,407.46)		
U2	58.3 X	28.105 X	0.0625 X	1 X	-0.5		(51.20)		38.867	(1,990.11)		
U3	4.6748 X	30.19 X	0.0625 X	1 X	-1		(8.82)		64.963	(573.01)		
U4	4.3252 X	2.0851 X	0.0625 X	1 X	-0.5		(0.28)		60.463	(17.04)		
U5	4.3252 X	28.105 X	0.0625 X	1 X	-1				61.183	(464.84)		
U6	0 X	73.53 X	0.0625 X	1 X	-1			0.00	0.000	0.00		
U7	0 X	28.105 X	0.0625 X	1 X	-0.5			0.00	0.000	0.00		
U8	0 X	30.19 X	0.0625 X	1 X	-1			0.00	0.000	0.00		
U9	0 X	2.0851 X	0.0625 X	1 X	-0.5			0.00	0.000	0.00		
U10	0 X	28.105 X	0.0625 X	1 X	-1			0.00	0.000	0.00		
HYDROSTATIC												
H1	103.72 X	103.72 X	0.0625 X	1 X	-0.5			(336.18)	34.573	(11,622.95)		
H2	73.5 X	73.5 X	0.0625 X	1 X	0.5			168.96	24.510	4,141.16		
MISC. HORIZONTAL												
E1	26.43 X	1 X	1.000 X	1 X	-1			(26.43)	32.280	(853.16)		
E2	20.47 X	1 X	1.000 X	1 X	1			20.47	35.380	724.23		
E3	X	X	0.000 X	1 X	-1				27.050			
E4	X	X	0.000 X	1 X	-1				27.050			
Sum V						<u>359.47</u>	Sum H		<u>(173.18)</u>	Sum M		<u>7,503.36</u>

$$M/V = 20.87 \text{ ft.}$$

$$e = M/V-B/2 = -12.78 \text{ ft.}$$

%Base in Compression = 93.1%

Sliding F.S.= 5.976
Bearing F.S.= 3.278

Max. Found. Pressure= 16.076 ksf
Bearing Capacity= 52.704 ksf

Wedge Analysis

FS =	5.976	Active Wedge	Passive Wedge	Structure Wedge
	0.950	$\alpha_a = -48.466$	$\alpha_p = 27.000$	$\alpha_s = 0.000$
Inputs				
Crossbed c =	17.500	$rad = -0.846$	$rad = 0.471$	$rad = 0.000$
Crossbed ϕ =	36.000	$Wa =$	$W_p = 18.771$	$W_s = 472.916$
$rad =$	0.628	$V_a =$	$V_p = 87.882$	$V_s = 87.931$
Unit Weight of Rock =	0.162	$U_a =$	$U_p = -16.681$	$U_s = -377.189$
c at Base =	1.800	$L_a =$	$L_p = 30.359$	$L_s = 67.300$
ϕ at Base =	25.000		$H_i - H_r = -7.573$	$H_i - H_r = 173.184$
$rad =$	0.436	$P_a = 0.000$	$P_p = 97.070$	$P_s = -97.070$
Top Rock U/S	857.00	Sum = 0.000		
Top Rock D/S	861.00			
Upper Pool	937.67			
Lower Pool	907.48			

Input Data

PROGRAM CSOILP - CALCULATION OF AT-REST PRESSURES BY COMBINATION
OF "GRAVITY-TURN-ON" AND THEORY OF ELASTICITY COMPONENTS
DATE: 10-AUGUST-06 TIME: 13:15:20

* INPUT DATA *

I.--HEADING
'Untitled'

II.--SURFACE POINTS

DIST. FROM WALL (FT)	ELEVATION (FT)
0.0	886.0

III.--SOIL LAYER DATA

<WEIGHT SAT.	(PCF)> MST.	INTERNAL FRICTION (DEG)	ELASTICITY COEFF.	BOTTOM ELEV. (FT)
125.0	120.0	35.0	0.00	880.0
128.0	125.0	0.0	0.00	857.0

IV.--WATER DATA

UNIT WEIGHT:	62.5 (PCF)
ELEVATION:	937.7 (FT)

Analysis Results

PROGRAM CSOILP - CALCULATION OF AT-REST PRESSURES BY COMBINATION
OF "GRAVITY-TURN-ON" AND THEORY OF ELASTICITY COMPONENTS
DATE: 10-AUGUST-06 TIME: 13:15:00

* RESULTS *

I.--HEADING
'Untitled'

II.--EQUIVALENT SURCHARGE LOAD DUE TO IRREGULAR SURFACE ABOVE 886.0 (FT)
NONE

III.--PRESSURES ON WALL BELOW EL. 886.0 (FT)

ELEVATION (FT)	GRAVITY- TURN-ON PRESSURE (PSF)	ELASTICITY COMPONENT PRESSURE (PSF)	COMBINED GTO & ELASTICITY PRESSURE (PSF)
886.00	0.00	0.00	0.00
885.00	26.65	0.00	26.65
884.00	53.30	0.00	53.30
883.00	79.95	0.00	79.95
882.00	106.61	0.00	106.61
881.00	133.26	0.00	133.26
880.00+	159.91	0.00	159.91
880.00-	375.00	0.00	375.00
879.00	440.50	0.00	440.50
878.00	506.00	0.00	506.00
877.00	571.50	0.00	571.50
876.00	637.00	0.00	637.00
875.00	702.50	0.00	702.50
874.00	768.00	0.00	768.00
873.00	833.50	0.00	833.50
872.00	899.00	0.00	899.00
871.00	964.50	0.00	964.50
870.00	1030.00	0.00	1030.00
869.00	1095.50	0.00	1095.50
868.00	1161.00	0.00	1161.00
867.00	1226.50	0.00	1226.50
866.00	1292.00	0.00	1292.00
865.00	1357.50	0.00	1357.50
864.00	1423.00	0.00	1423.00
863.00	1488.50	0.00	1488.50
862.00	1554.00	0.00	1554.00
861.00	1619.50	0.00	1619.50
860.00	1685.00	0.00	1685.00
859.00	1750.50	0.00	1750.50
858.00	1816.00	0.00	1816.00
857.00+	1881.50	0.00	1881.50

Resultant Force for Combined Pressures = 26429.48 lbs
Resultant Location at elev. = 866.23 feet

Input Data

PROGRAM CSOILP - CALCULATION OF AT-REST PRESSURES BY COMBINATION
OF "GRAVITY-TURN-ON" AND THEORY OF ELASTICITY COMPONENTS
DATE: 10-AUGUST-06 TIME: 13:17:14

* INPUT DATA *

I.--HEADING

II.--SURFACE POINTS

DIST. FROM WALL (FT)	ELEVATION (FT)
0.0	886.0

III.--SOIL LAYER DATA

<WEIGHT (PCF)>	SAT.	MST.	INTERNAL FRICTION (DEG)	ELASTICITY COEFF.	BOTTOM ELEV. (FT)
128.0	125.0		0.0	0.00	861.0

IV.--WATER DATA

UNIT WEIGHT:	62.5 (PCF)
ELEVATION:	907.5 (FT)

Analysis Results

PROGRAM CSOILP - CALCULATION OF AT-REST PRESSURES BY COMBINATION
OF "GRAVITY-TURN-ON" AND THEORY OF ELASTICITY COMPONENTS
DATE: 10-AUGUST-06 TIME: 13:16:56

* RESULTS *

I.--HEADING

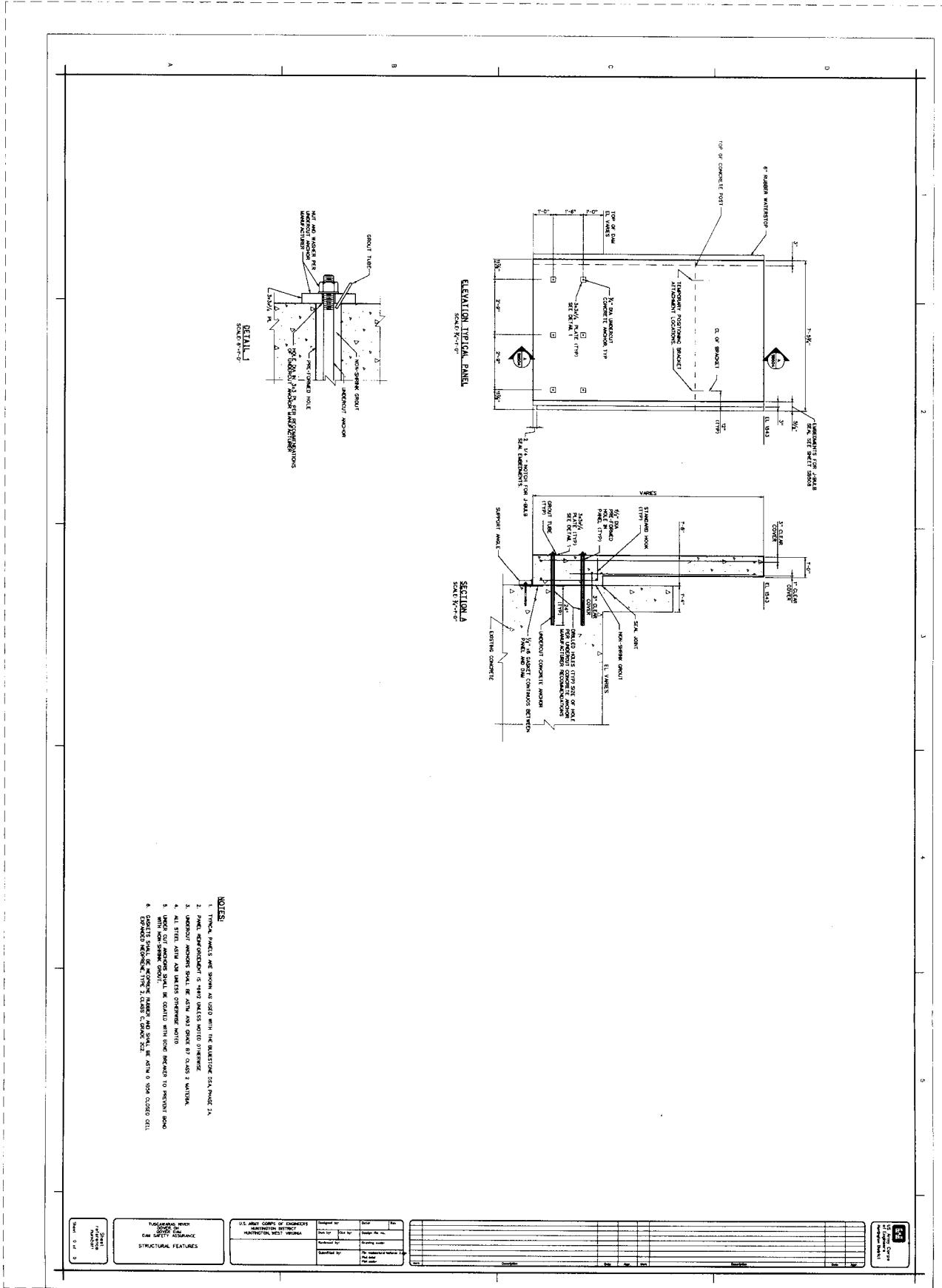
II.--EQUIVALENT SURCHARGE LOAD DUE TO IRREGULAR SURFACE ABOVE 886.0 (FT)
NONE

III.--PRESSURES ON WALL BELOW EL. 886.0 (FT)

ELEVATION (FT)	GRAVITY- TURN-ON PRESSURE (PSF)	ELASTICITY COMPONENT PRESSURE (PSF)	COMBINED GTO & ELASTICITY PRESSURE (PSF)
886.00	0.00	0.00	0.00
885.00	65.50	0.00	65.50
884.00	131.00	0.00	131.00
883.00	196.50	0.00	196.50
882.00	262.00	0.00	262.00
881.00	327.50	0.00	327.50
880.00	393.00	0.00	393.00
879.00	458.50	0.00	458.50
878.00	524.00	0.00	524.00
877.00	589.50	0.00	589.50
876.00	655.00	0.00	655.00
875.00	720.50	0.00	720.50
874.00	786.00	0.00	786.00
873.00	851.50	0.00	851.50
872.00	917.00	0.00	917.00
871.00	982.50	0.00	982.50
870.00	1048.00	0.00	1048.00
869.00	1113.50	0.00	1113.50
868.00	1179.00	0.00	1179.00
867.00	1244.50	0.00	1244.50
866.00	1310.00	0.00	1310.00
865.00	1375.50	0.00	1375.50
864.00	1441.00	0.00	1441.00
863.00	1506.50	0.00	1506.50
862.00	1572.00	0.00	1572.00
861.00+	1637.50	0.00	1637.50

Resultant Force for Combined Pressures = 20468.75 lbs
Resultant Location at elev. = 869.33 feet

Dover Parapet Wall



Dover Parapet and I-wall quantitiesBY: ECV
5-Sep-06**Alternative 1**

Parapet Wall	Location	height (ft)	length (ft)
Left Abutment			
	End wall	9	25
	STA. 7+00 to 8+44	9	144
	STA. 8+44 to 9+21	7.5	77
Right Abutment			
	End wall	9	25
	Spillway to Operations Building	9	18
	Operations building to I-wall	9	10
	Face of Operations building with one foot overlap on ends	9	19
 I-wall Assume base EL 932.0			
Left Abutment			
		8	136
Right Abutment			
		8	178

Parapet Wall Quantities - Alt 1BY: ECV
5-Sep-06

Length of Wall

241 ft of	9.0' Wall
77 ft of	7.5' Wall

Assume joints every 10 ft

Wall Height (FT)	Quantities per linear foot		Concrete Removal (CY)	Y stop (LF per joint)
	Concrete (CY)	Re-Steel (LB)		
9 FT	0.50	63	0.18	13
	241	241	241	25
TOTAL	121	15,144	44	325

Wall Height (FT)	Quantities per linear foot		Concrete Removal (CY)	Y stop (LF per joint)
	Concrete (CY)	Re-Steel (LB)		
7.5 FT	0.39	49	0.15	10.5
	77	77	77	8
TOTAL	30	3,765	11.4	84

I-Wall Quantities

Length of Wall

314 ft of 8' Wall

Assume joints every 10 ft

Wall Height (FT)	Quantities per linear foot					
	Concrete (CY)	Common Excavation (CY)	Re-Steel (LB)	Backfill (CY)	LF Sheet Piling (SF)	Y stop (LF per joint)
8 FT	0.845	0.93	105.7	0.61	20	7
	314	314	314	314	314	32
TOTAL	265	292	33,190	192	6,280	224

ALT 1 - SUMMARY

	Concrete (CY)	Common Excavation (CY)	Re-Steel (LB)	Backfill (CY)	LF Sheet Piling (SF)	Y stop (LF per joint)
Total	417	292	52,099	192	6,280	633



**US Army Corps
of Engineers**
Huntington District

**Dover DSA
Quantities**

Rt. 800 Swing Gate A/H. **COMPUTATIONS**
I-wall Quantities
(not including parapet wall)

Computed By: EZV

Date 5 SEPT 06

Checked By: JW

Date _____

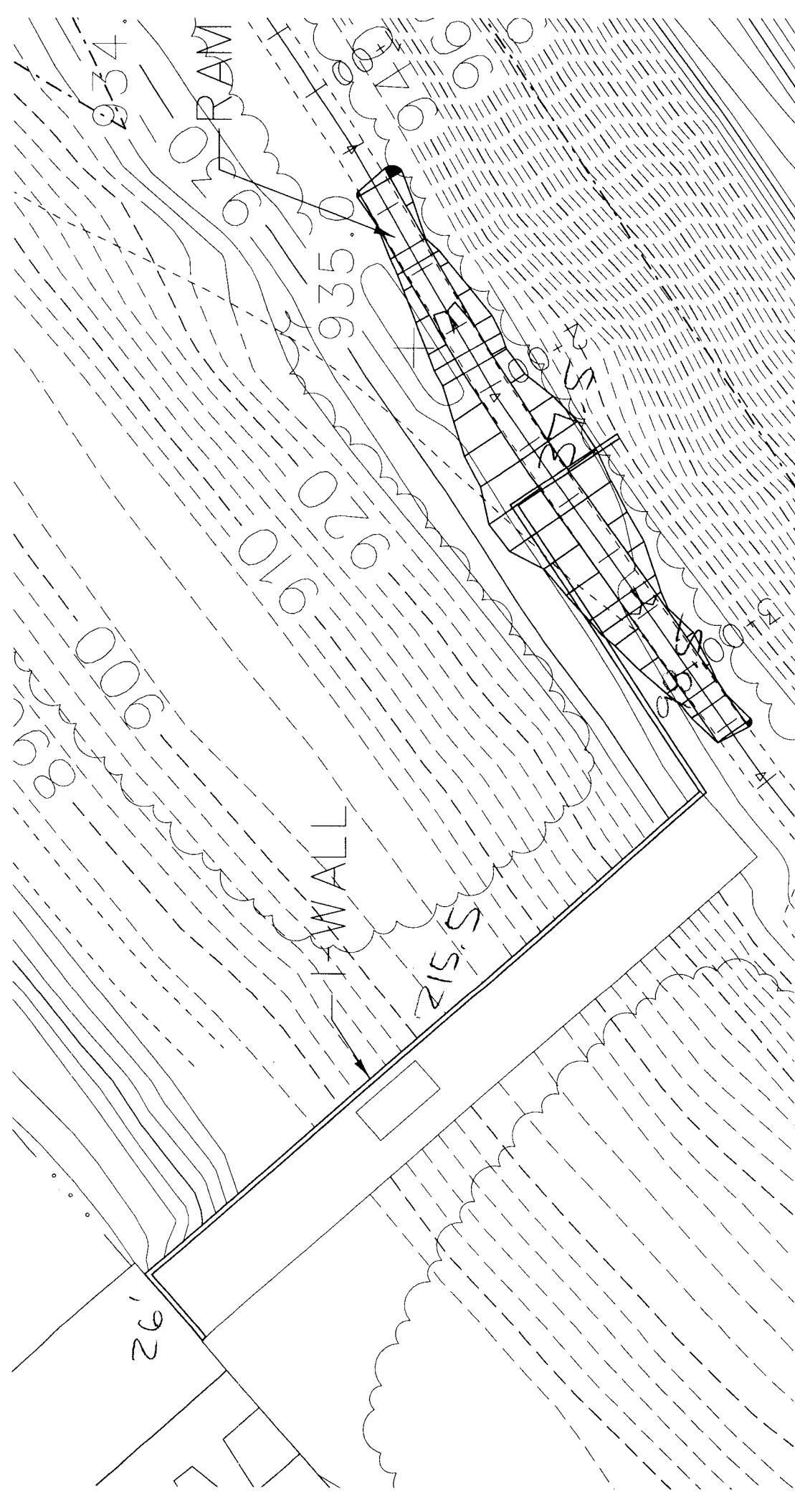
LEFT ABUTMENT	LENGTH	HEIGHT
	98.5	8'
	37.5	8'
	136'	/

RIGHT ABUTMENT	LENGTH	HEIGHT
	71.5	8'
	21	8'
	85.5	8'
	178'	/

TOTAL $136 + 178 = 314'$

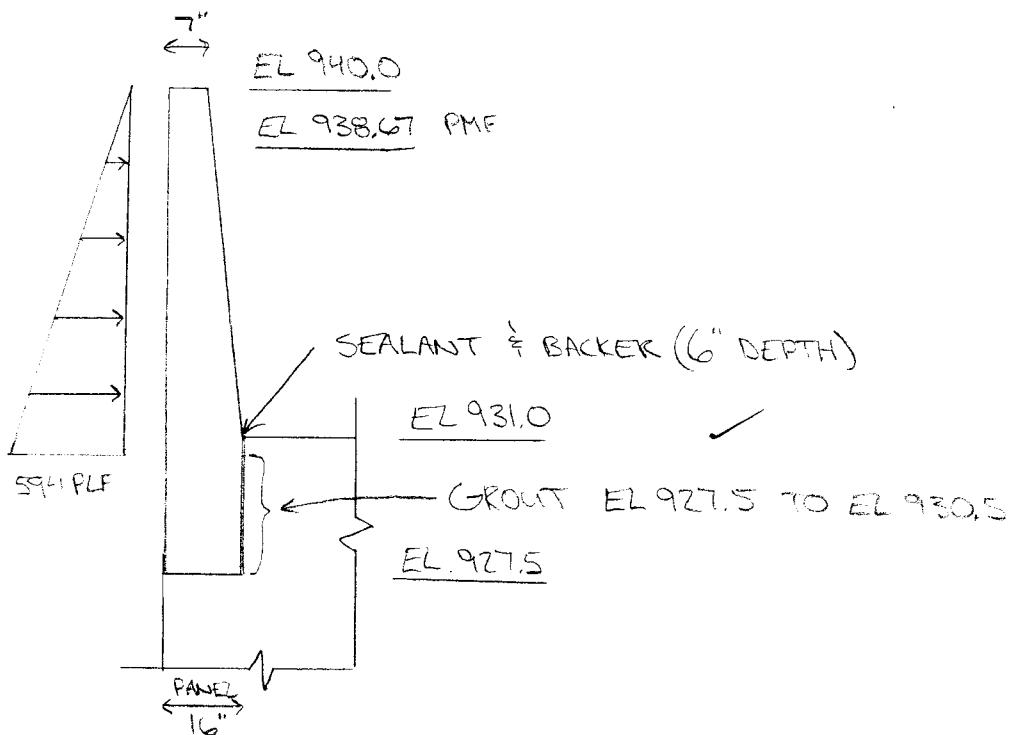


Dover - Left Monument



PARAPET WALL

- DESIGN FOR FULL STATIC LOAD -
- ASSUME BASE OF WALL IS FIXED AT EL 930.5 -
- SHAPE : US SIDE IS VERTICAL,
DS SIDE IS TAPERED.
- CUT THE WALL BASE INTO MONOLITH ($\frac{1}{3}$ OF EXPOSED HEIGHT)



$$V_u = 1.3 \times 1.7 \times [62.5 \text{ psf} \times 1' \times 9.5 \frac{3}{4}] = 6233 \#$$

$$M_u = 6233 \times 9.5 \frac{3}{4} \times 12 / 1000 = 236.8 \text{ k-in}$$

PARAPET WALL

DOVER DSA

AUGUST 2006 ECV

✓ SAW

MIN THICKNESS BASED ON SHEAR

$$\phi V_c > V_u = 0.35 \times 2\sqrt{3000} \times 12 \times d$$

$$d = \frac{6233 \times 2}{0.35 \times 2\sqrt{3000} \times 12} = 11.16"$$

$$\text{MIN THICKNESS} = 11.16 + 3" = 14.16$$

USE 16" BASE THICKNESS ✓

 $A_{s,\text{req'd}}$ 9.0' WALL

$$\text{VERTICAL BARS } A_{s,\text{req'd}} = 0.47 \text{ in}^2/\text{ft}$$

$$\text{USE } \#6 @ 10" \quad A_s = 0.53 \text{ in}^2/\text{ft} \quad -$$

$$\text{HORIZONTAL BARS } A_{s,\text{req'd}} = 0.17 \text{ in}^2/\text{ft}$$

$$\text{USE } \#4 @ 12" \quad A_s = 0.20 \text{ in}^2/\text{ft} \quad -$$

7.5' WALL

$$\text{VERTICAL BARS } A_{s,\text{req'd}} = 0.30 \quad \text{USE } \#6 @ 10" \quad -$$

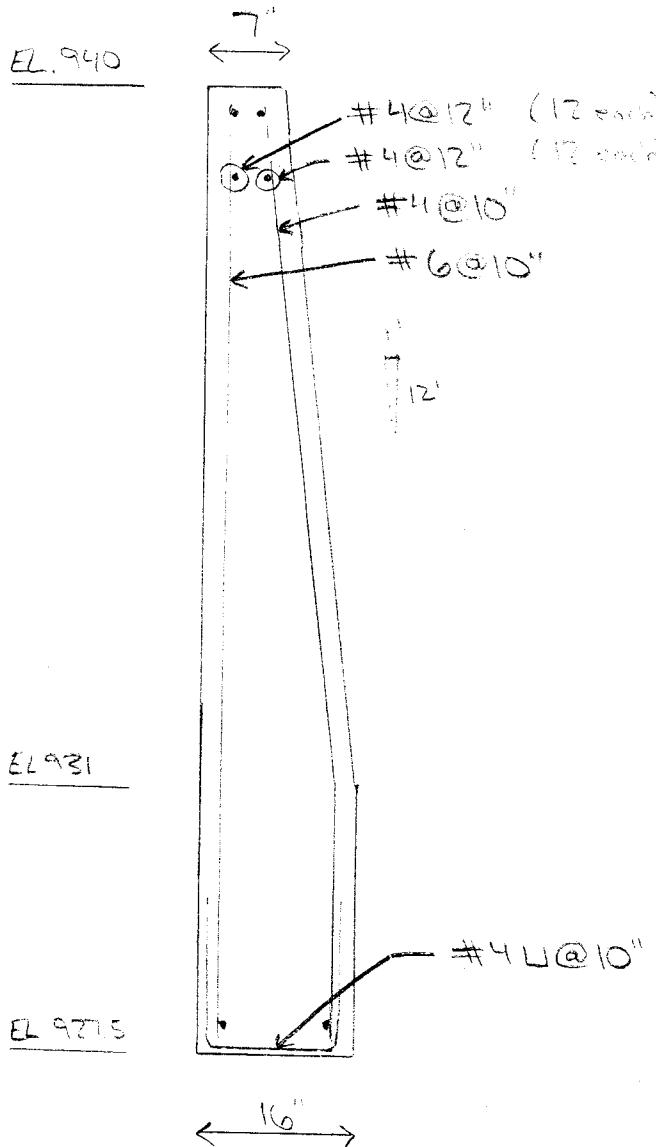
$$\text{HORIZONTAL BARS } A_{s,\text{req'd}} = 0.16 \quad \text{USE } \#4 @ 12" \quad -$$

PARAPET WALL

DOVER DSA

AUGUST 2006 EKV

✓/SU

9.0' EXPOSED HEIGHT PARAPET WALL

$$\text{CONCRETE : AREA} = \left[(3.5 \times 16) + \left(\frac{7+16}{2} \times 9 \right) \right] / 27 = 0.50$$

$$\text{VOLUME} = 0.50 \text{ CY/FT} \quad \checkmark$$

$$\text{REINFORCEMENT : } 125 \text{ LB/CY} \times 0.50 \text{ CY/FT} = 62.5 \text{ LB/FT} \quad \checkmark$$

PARAPET WALL

DOVER DSA

AUGUST 2006 ECV

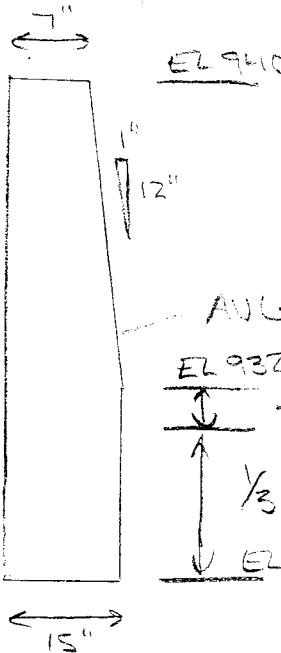
✓ Skw

LEFT ABUTMENT

STA 7+00 TO 8+44 \Rightarrow 144' EL 931.0STA 8+44 TO 9+21 \Rightarrow 77' 3.7% SLOPE
TO 933.95 \hookrightarrow AVG HEIGHT = 932.5 \hookrightarrow RAISE BASE EL 1' EACH $[1/2 \times 37] = 27'$ BASE THICKNESS

16"	EL 931
15"	EL 932
14"	EL 933

LENGTH	/
27'	/
27'	/
23'	/



N.T.S.

Avg. CONCRETE : $[(2.5 \times 15/2) + (\frac{7+15}{2} \times 7.5)]/27 = 10\%$
 $= 0.37 \text{ CY/FT}$

REINFORCEMENT : $125 \text{ LB/CY} \times 0.37 \text{ CY/FT} = 46.3 \text{ LB/FT}$

	Exposed Wall Height (ft)	thickness, min (in)	thickness (in)	Cover (in)	$A_{s, \text{req'd}}$ per ft	Transverse	US Vertical	DS Vertical	Horizontal E.F.
base	9.5	14.2	16	3	0.47	0.17	#6@10	#4@10	#4@12
base	7.5	10.9	15	3	0.30	0.16	#6@10	#4@10	#4@12
top	0		7	3					

STEEL REINFORCEMENT CALCULATION

✓ GW

M _u =	236.8 k-in / linear foot	Exposed Wall Height =	9 ft
V _u =	6233 lb	Depth to grout =	0.5 ft
b =	12 in	Static Head =	9.5 ft
d =	12.81 in		
t =	16 in	Clear Cover =	3 in
f' _c =	3 ksi	#6 bar =	0.38 in
f _y =	60 ksi		
β_1 =	0.85 Unitless		
P _{u axial} =	0 Kips (+ compression, - tension)		
A _g =	192 in ²		

Calculations

All Areas are per side

If axial load is present a value must be found for Φ

Use Φ = 0.900

Calculate M_n and P_n

M_n = 263.17 k-in / linear foot

P_n = 0.00 Kips

K_u = 0.054

A_{s Req'd} = 0.35 in²

A_{s min} = 0.35 in² 0.0018 b t (ACI 318-05 7.12.2 Temp and Shrinkage Structural Slab Resteel required at right angles to principal resteel)
total for both faces

A _{s min} =	0.42 in ²	Not less than (3 √ (f' _c) / F _y) b d	(ACI 318-05 10.5.1 Flexure)
A _{s min} =	0.51 in ²	Not less than 200 b d / F _y (psi)	(ACI 318-05 10.5.1 Flexure Minimum)
A _{s min} =	0.47 in ²	Unless 1.33 (A _{s Req'd})	(ACI 318-05 10.5.3 Flexure - If A _{smin} >A _{sreqd})

$A_{st,s} = 0.27 \text{ in}^2$ $0.0014 b t$ (EM 1110-2-2104, 2-8 Temp and Shrinkage
.0028 x gross cross sectional area.
half in each face - .0014)

$A_{s,reqd} = 0.47 \text{ in}^2$ **Governs**

USE $A_s = 0.53 \text{ in}^2$ (Input by Designer)
 $\#6 @10"$

Check Moment

Calculate ΦM_n and compare to Original M_u

$$\Phi M_n = 351.82 \text{ k-in} = 0.9 A_s F_y d (1 - (A_s F_y / 2 \beta_b d f'_c))$$

$$M_u = 236.8498438 \text{ k-in / linear foot}$$

If $\Phi M_n > M_u$ then A_s is acceptable **OK**

Check Shear (ACI 318-05 11.3)

Calculate ΦV_c and Compare to V_u

$$\Phi V_c = 14316 \text{ Lbs} = 0.85 \times 2 (\sqrt{f'_c}) b d$$

$$V_u = 6232.890625 \text{ Lbs}$$

If $\Phi V_c > V_u$ then A_s is acceptable **OK**

Check for Stirrups (ACI 318-05 11.5.6)

Stirrups are required if $V_u > .5 \Phi V_c$ **Stirrups not Needed**

Check Reinforcement Ratio (EM 1110-2-2104, 3-5 Max Tension Reinforcement)

$$\rho_{actual} = 0.00345 \quad \rho_{actual} = A_s / b d$$

$$\rho_b = 0.0214 \quad \rho_b = .85 \beta_1 (f'_c / f_y) (87000 / (87000 + f_y))$$

$$0.25 \rho_b = 0.0053 \quad \text{Recommended Limit}$$

$$0.375 \rho_b = 0.0080 \quad \text{Maximum Upper limit without special study}$$

$\rho_{actual} \leq 0.25 \rho_b$ (Recommended) **OK**

$\rho_{actual} \leq 0.375 \rho_b$ (Always) **OK**

STEEL REINFORCEMENT CALCULATION

$M_u =$	141.4 k-in / linear foot	Exposed Wall Height =	7.5 ft
$V_u =$	4420 lb	Depth to grout =	0.5 ft
$b =$	12 in	Static Head =	8 ft
$d =$	11.81 in		
$t =$	15 in	Clear Cover =	3 in
$f'_c =$	3 ksi	#6 bar =	0.38 in
$f_y =$	60 ksi		
$\beta_1 =$	0.85 Unitless		
$P_{u_axial} =$	0 Kips (+ compression, - tension)		
$A_g =$	180 in ²		

Calculations

All Areas are per side

If axial load is present a value must be found for Φ

Use $\Phi =$ 0.900

Calculate M_n and P_n

$M_n =$	157.16 k-in / linear foot		
$P_n =$	0.00 Kips		
$K_u =$	0.038		
$A_{s_Reqd} =$	0.23 in ²		
$A_{s_min} =$	0.32 in ²	0.0018 b t	(ACI 318-05 7.12.2 Temp and Shrinkage Structural Slab Resteel required at right angles to principal resteel)

$A_{s_min} =$	0.39 in ²	Not less than $(3 \sqrt{f'_c} / F_y) b d$	(ACI 318 -05 10.5.1 Flexure)
$A_{s_min} =$	0.47 in ²	Not less than $200 b d / F_y$ (psi)	(ACI 318-05 10.5.1 Flexure Minimum)
$A_{s_min} =$	0.30 in ²	Unless $1.33 (A_{s_Reqd})$	(ACI 318-05 10.5.3 Flexure - If $A_{smin} > A_{sreqd}$)

$A_{st\&s} = 0.25 \text{ in}^2$ $0.0014 b t$ (EM 1110-2-2104, 2-8 Temp and Shrinkage
.0028 x gross cross sectional area.
half in each face - .0014)

$A_{s\text{ reqd}} = 0.30 \text{ in}^2$ **Governs**

USE $A_s = 0.53 \text{ in}^2$ (Input by Designer)
 $\#6 @ 10"$

Check Moment

Calculate ΦM_n and compare to Original M_u

$$\Phi M_n = 323.20 \text{ k-in} = 0.9 A_s F_y d (1 - (A_s F_y / 2 \beta_b d f'_c))$$

$$M_u = 141.44 \text{ k-in / linear foot}$$

If $\Phi M_n > M_u$ then A_s is acceptable **OK**

Check Shear (ACI 318-05 11.3)

Calculate ΦV_c and Compare to V_u

$$\Phi V_c = 13199 \text{ Lbs} = 0.85 \times 2 (\sqrt{f'_c}) b d$$

$$V_u = 4420 \text{ Lbs}$$

If $\Phi V_c > V_u$ then A_s is acceptable **OK**

Check for Stirrups (ACI 318-05 11.5.6)

Stirrups are required if $V_u > .5 \Phi V_c$ **Stirrups not Needed**

Check Reinforcement Ratio (EM 1110-2-2104, 3-5 Max Tension Reinforcement)

$$\rho_{actual} = 0.00374 \quad \rho_{actual} = A_s / b d$$

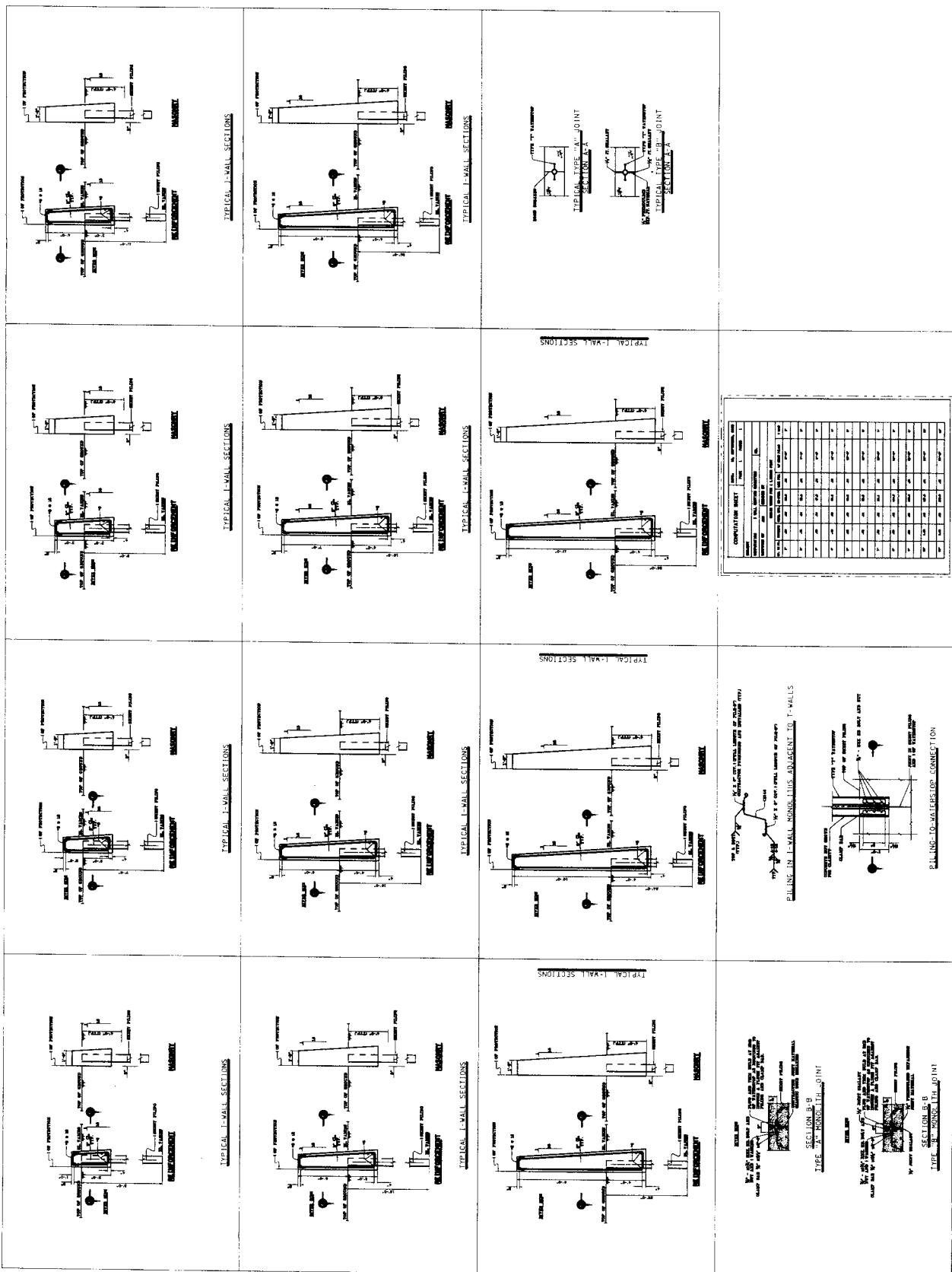
$$\rho_b = 0.0214 \quad \rho_b = .85 \beta_1 (f'_c / f_y) (87000 / (87000 + f_y))$$

$$0.25 \rho_b = 0.0053 \quad \text{Recommended Limit}$$

$$0.375 \rho_b = 0.0080 \quad \text{Maximum Upper limit without special study}$$

$\rho_{actual} \leq 0.25 \rho_b$ (Recommended) **OK**

$\rho_{actual} \leq 0.375 \rho_b$ (Always) **OK**



SLOPED SPILLWAY ALTERNATIVE AND QUANTITIES

Allows water to overtop the abutments and flow into two sloped spillways that lead to the stilling basin.

PMF (Overflow Dam) EL 937.5
Tail water EL 907.5

Quantities - Alt 2BY: ECV
5-Sep-06

✓ JFW

Sloped Spillway

Concrete	434.1 CY
Rebar	54263 LB

I-Wall Quantities

Length of Wall

378 ft of 8' Wall

Assume joints every 10 ft

Wall Height (FT)	Quantities per linear foot					Y stop (LF per joint)
	Concrete (CY)	Common Excavation (CY)	Re-Steel (LB)	Backfill (CY)	LF Sheet Piling (SF)	
8 FT	0.845	0.93	105.7	0.61	20	7
	378	378	378	378	378	38
TOTAL	319	352	39,955	231	7,560	266

ALT 2 - SUMMARY

	Concrete (CY)	Common Excavation (CY)	Re-Steel (LB)	Backfill (CY)	LF Sheet Piling (SF)	Y stop (LF per joint)
Total	754	352	94,218	231	7,560	266

Dover sloped spillway quantitiesBY: ECV
5-Sep-06**Alternative 2**

	Location	height (ft)	length (ft)
Parapet Wall	None		
Sloped Spillway	see hand calcs		

I-wall	Assume base EL 932.0		
Left Abutment		8	166.5
Right Abutment		8	211.5



**US Army Corps
of Engineers**
Huntington District

Dover DSA Quantities

SLOPED SPILLWAY ALT. COMPUTATIONS
I-WALL QUANTITIES
(NOT INCLUDING PARAPET WALL)

Computed By: ECU

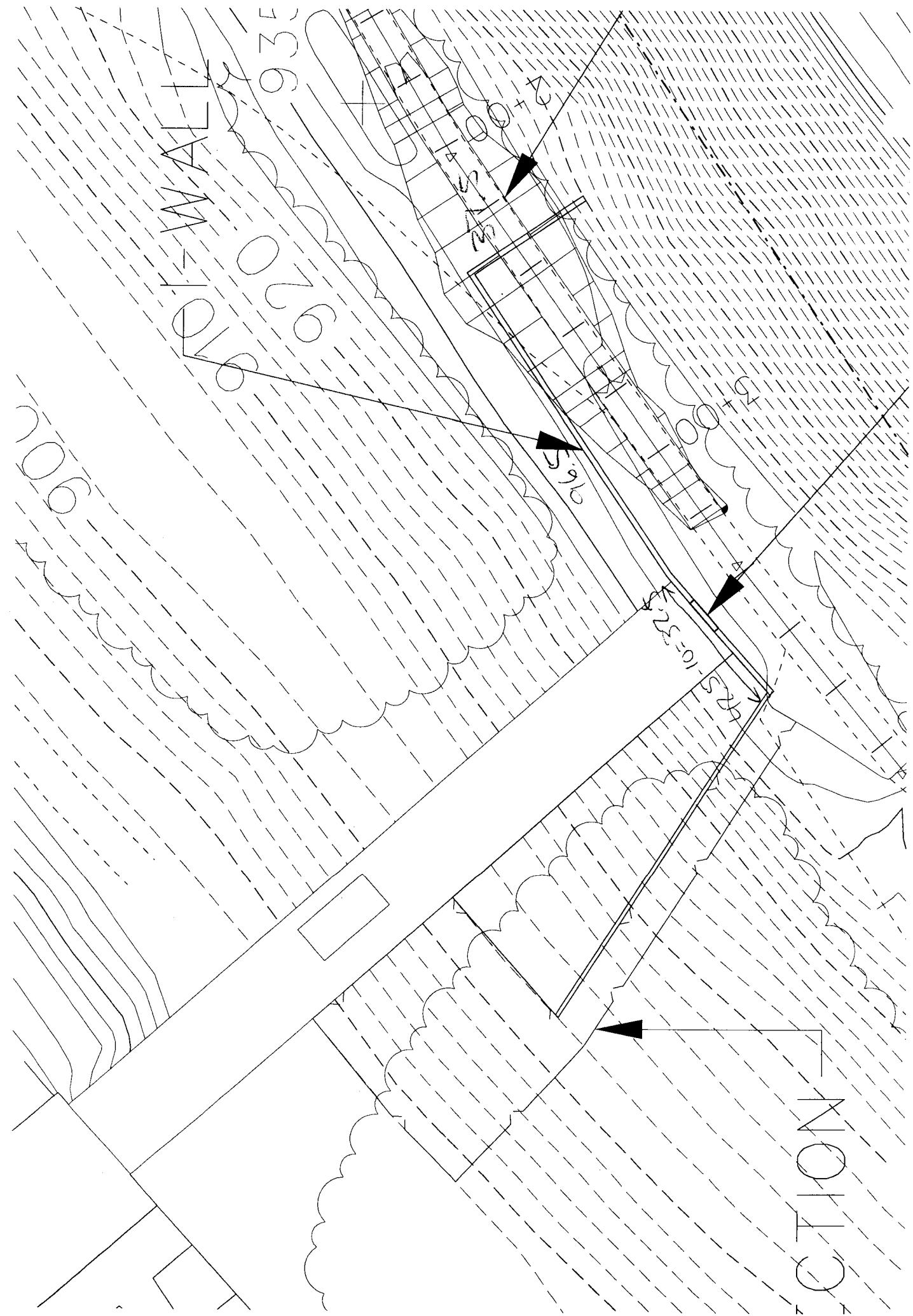
Date 5 SEPT 06

Checked By: Sar

Date

ABUTMENT		LENGTH	HEIGHT				
LEFT ABUTMENT		32.5'	8'				
		96.5'	8'				
		37.5'	8'				
RIGHT ABUTMENT		15					
		7.5					
		2					
		2					
		9					
		71					
		21					
		60.5					
		23.5					
TOTAL		166.5 + 211.5 = 378					

LEFT ABUTMENT

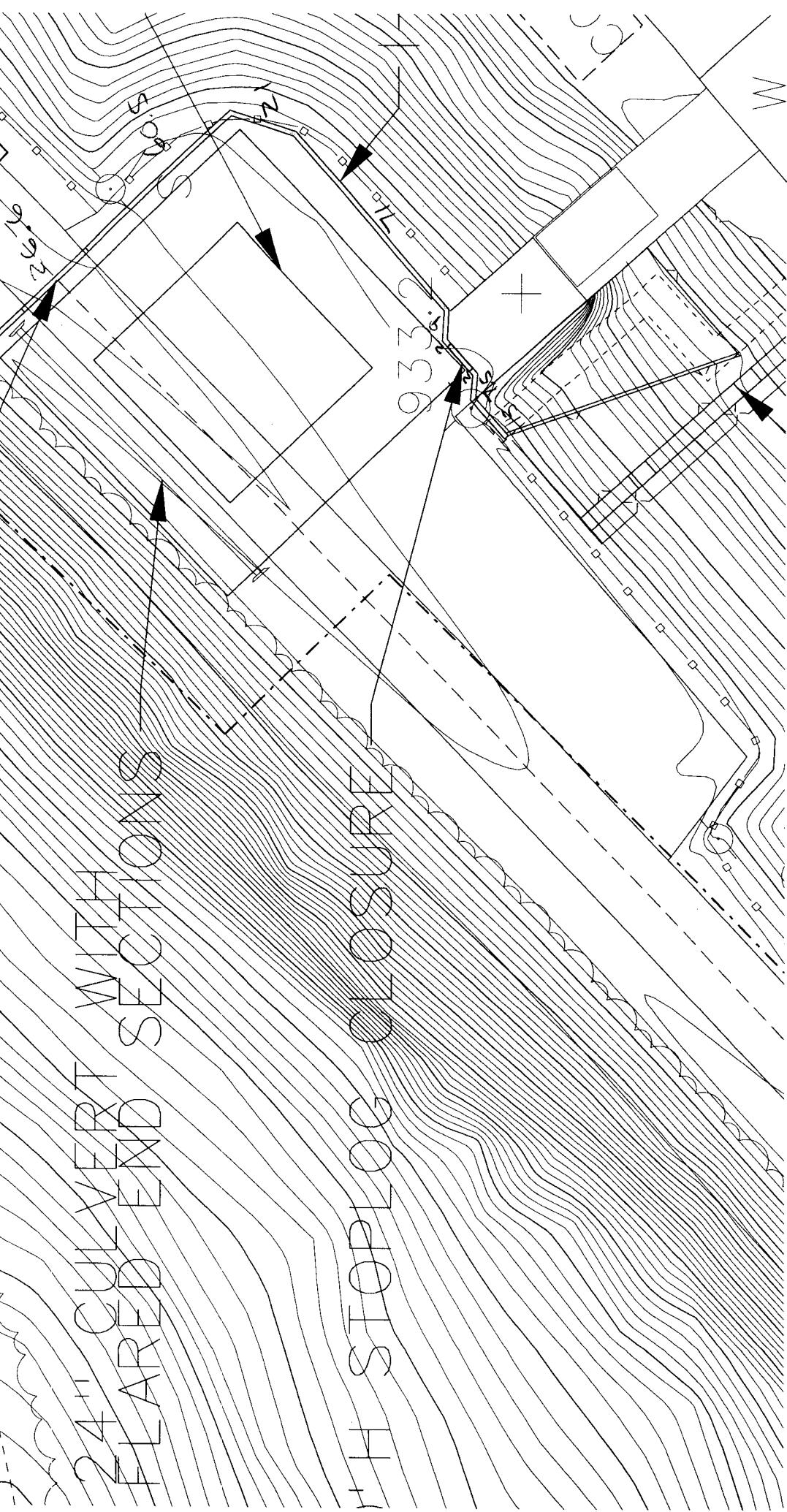


RIGHT ALIGNMENT

SWING GATE

FLARED CULVERT SECTION

STOP LOG GROOVE





**US Army Corps
of Engineers**
Huntington District

**Dover DSA
Quantities**

ANGLED SPILLWAY

COMPUTATIONS

Computed By: ECU

Date 8/21/06

Checked By: SMW

Date _____

RIGHT ABUTMENT	LENGTH	AVG WIDTH	HEIGHT		VOLUME		
BASE SLAB - CONCRETE	147.3'	<u>14+42</u> <u>2</u>	1'	/27	152.8 CY		
WALL - CONCRETE	147.3'	1'	10'+1'	/27	60.0 CY		
					212.8 CY ✓		
REINFORCEMENT	125 LB/CY	212.8	26600 LB	/			
LEFT ABUTMENT	LENGTH	AVG WIDTH	HEIGHT		VOLUME		
BASE SLAB - CONCRETE	153.2'	<u>14+42</u> <u>2</u>	1'	/27	158.9		
WALL CONCRETE	153.2'	1'	10'+1'	/27	62.4		
					221.3 ✓		
REINFORCEMENT	125 LB/CY	221.3	27663 LB ✓				
TOTAL CONCRETE	434.1 CY ✓						
REINFORCEMENT	541263 LB						

ANGLED SPILLWAY DOVER DSA

ECU

8/21/06

✓spw

TOP OF DAM = 931.5 TO 933.95

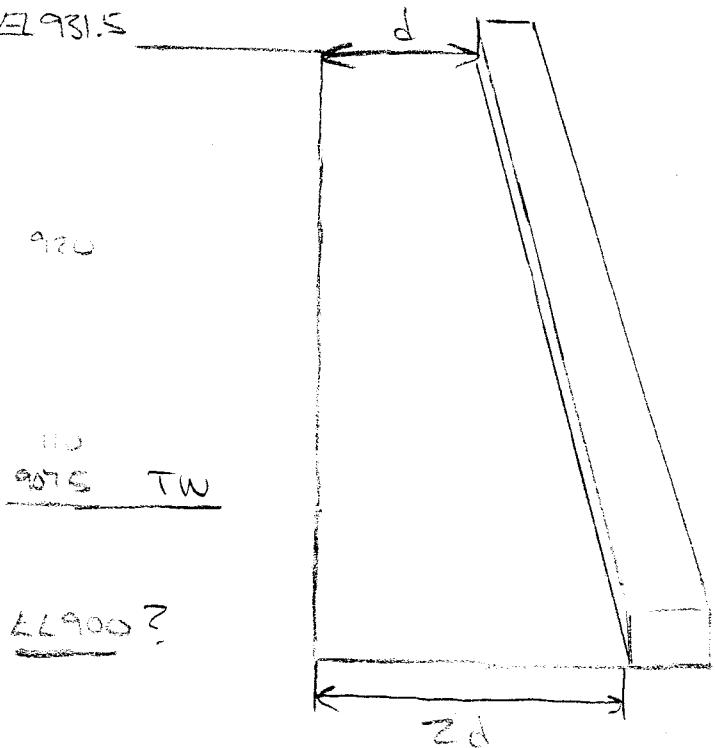
PMF TW = 907.48

PMF HW (OVERFLOW OPTION) = 937.5

EL 937.5

H = 30' MAX

EL 931.5



Q = 5000 CFS

FOR NON-OVERFLOW
AREAS
- FROM TED HAMB

AREA FROM MICROSTATION

LEFT ABUTMENT 1571 SF

RIGHT ABUTMENT 10412 SF

2613 SF

ADDITION AREA DEDUCTIONS
ON THE NEXT PAGE

SIDE VIEW

ASSUME SLAB 12"

ASSUME WALL 12"

VERTICAL

$$d = \sqrt{t^2 + \frac{1}{2} g t^2}$$

$$30' = \frac{1}{2} g t^2$$

$$t = \sqrt{\frac{2 \times 30'}{\frac{1}{2} g}} = 1.365 \text{ sec}$$

$$V = V_0 + at$$
$$t = \sqrt{t^2 + 0.5at^2}$$

$$\text{HORIZONTAL } d = vt + \frac{1}{2} \sqrt{t^2}$$

$$d = vt$$

$$d = (2.64(\frac{7}{8}))(1.365) = 3.6'$$

(Low: CHECK VALUE)

AUG VELOCITY IN ANGLED SPILLWAY (DOWN SLOPE ONLY)

$$\frac{V_1^2}{2g} + \frac{P_1}{\gamma w} + z_1 = \frac{V_2^2}{2g} + \frac{P_2}{\gamma w} + z_2 + h_L$$

$$(z_1 - z_2) = \frac{V_2^2}{2g}$$

$$V = \sqrt{2g(z_1 - z_2)}$$

$$@ 15' V = \sqrt{2(32.2) 15'} = 31.1 \text{ f/s}$$

$$@ 30' V = \sqrt{2(32.2) 30'} = 44.0 \text{ f/s}$$

(WITHOUT LOSSES)

DISCHARGE WILL HAVE AN INITIAL HORIZONTAL VELOCITY DUE TO THE HORIZONTAL COMPONENT OF THE DISCHARGE IMPACTING THE SLOPED SPILLWAY.

$$Q_i = VA = 9.8 \text{ f/s} \times 1240 \text{ SF} = 12152 \text{ CFS}$$

$$Q_{out} = 44 \text{ f/s} \times (3 \times 11) \times 10' = 13480 \text{ CFS}$$

NO LOSSES INCLUDED \rightarrow USE 10' WALL ✓

ANGLED SHILLWAY DOVER DSA

ECV

8/21/06
✓ SAW

NON-OVERFLOW PMF WATER AREA

BOTH ABUTMENTS $1571 + 1042$

- OPERATING HOUSE = $34' \times (937.5 - 931) = 221 \text{ SF}$

- ENTRANCE HOUSE = $25' \times (937.5 - 931) = 162 \text{ SF}$

- CONCRETE POSTS = 337

AREA = $1571 + 1042 - 221 - 162 - 337 = 1893 \text{ SF}$

$$V = \frac{5000 \text{ CFS}}{1893 \text{ SF}} = 2.64 \text{ FT/S (LOW)} \quad \checkmark$$

CONCRETE POSTS

$$\text{AREA} = 2.75' \times 3.5' + 0.75' \times 2.42' = 10.73 \text{ SF}$$

$$10.73 \times (19+7) = 265.98$$

$$(R, Ab 10\%) + 2' \times 1.5' \times (2) = 6$$

$$(L, H, Rds) + 3' \times 6.5' + 3' \times 5.15' + 6.5 \times 3' + 3' \times 3.55 = 65.1$$

$$\text{TOTAL} = 265.98 + 6 + 65.1 = 337 \quad \checkmark$$

$$\text{RIGHT ABUTMENT } 1042 - 221 - \frac{337}{2} = 653 \quad \checkmark$$

$$\text{LEFT ABUTMENT } 1571 - 162 - \frac{337}{2} = 1240 \quad -$$

ANGLE SPILLWAY DOVER DSA ECV ✓ 8/21/06
CHECK HORIZONTAL WATER VELOCITY ✓ SW

$$\frac{V_1^2}{2g} + \frac{\rho g}{w} + z_1 = \frac{V_2^2}{2g} + \frac{\rho g}{w} + z_2 + h_L$$

$$V_{\max} = \sqrt{2(32.2) \times 6'} = \sqrt{2(32.2) \times 6'} = 19.6 \text{ ft/s}$$

$$V_{\text{AUG (no waves)}} = 9.8 \text{ ft/s} \quad (\text{TED HAMM AGREES}) \quad 8/21/06$$

$$d = vt + \frac{1}{2} \alpha t^2 = 9.8 \text{ ft/s} \times 1.365 \text{ sec} = 13.4 \text{ ft} \quad \checkmark$$

SAY 14 FT

LENGTH OF SPLASH PAD

RIGHT ABUTMENT

EL 933 TO EL 902

$$\text{LENGTH} = \text{STA } 3+26 - \text{STA } 1+82 = 144'$$

$$\text{AVG SLOPE } \frac{933-902}{144} = 21.5\% \text{ OR } 2.55 \frac{\text{IN}}{\text{FT}}$$

$$\text{LENGTH} = \sqrt{31^2 + 144^2} = 147.3' /$$

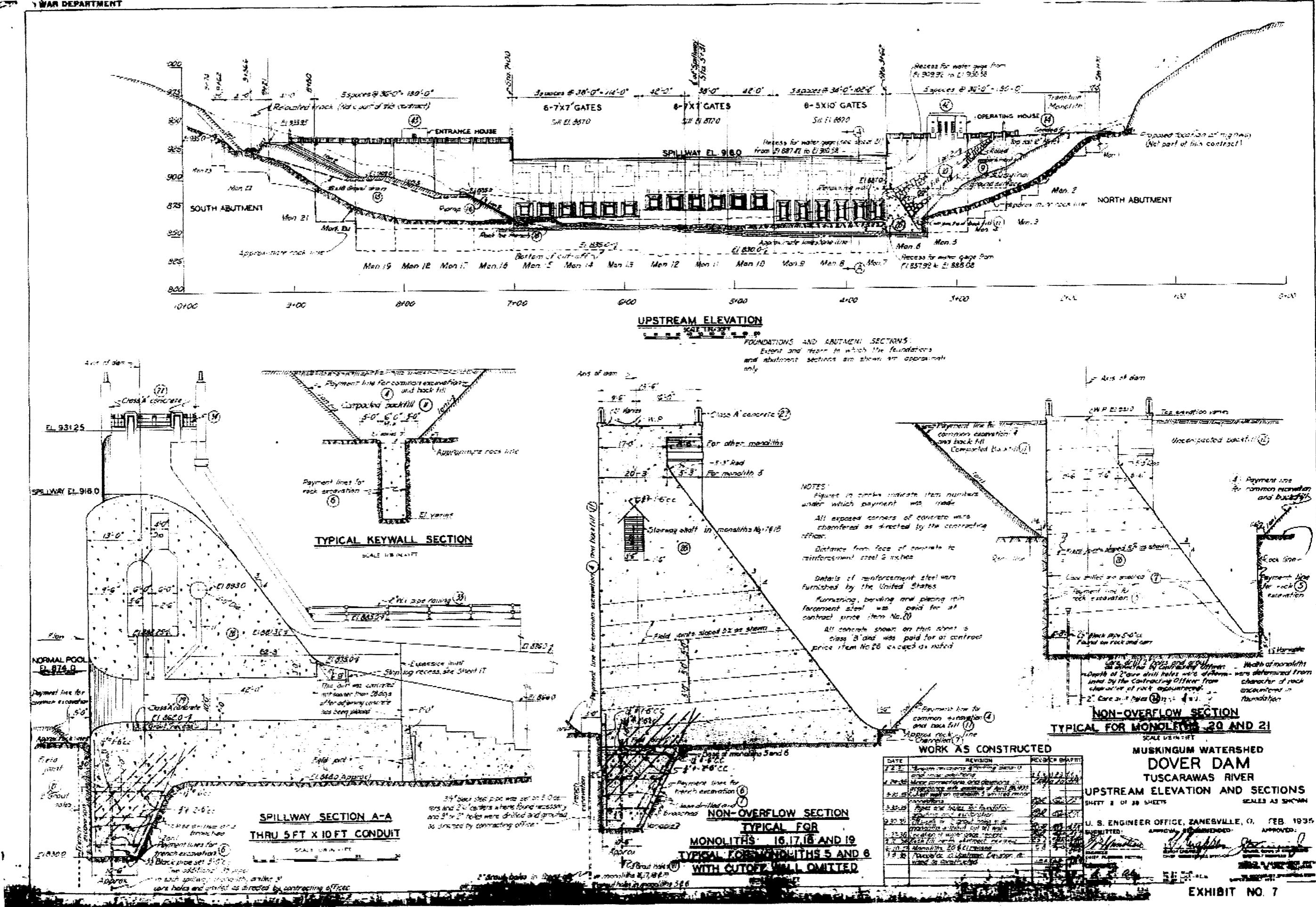
LEFT ABUTMENT

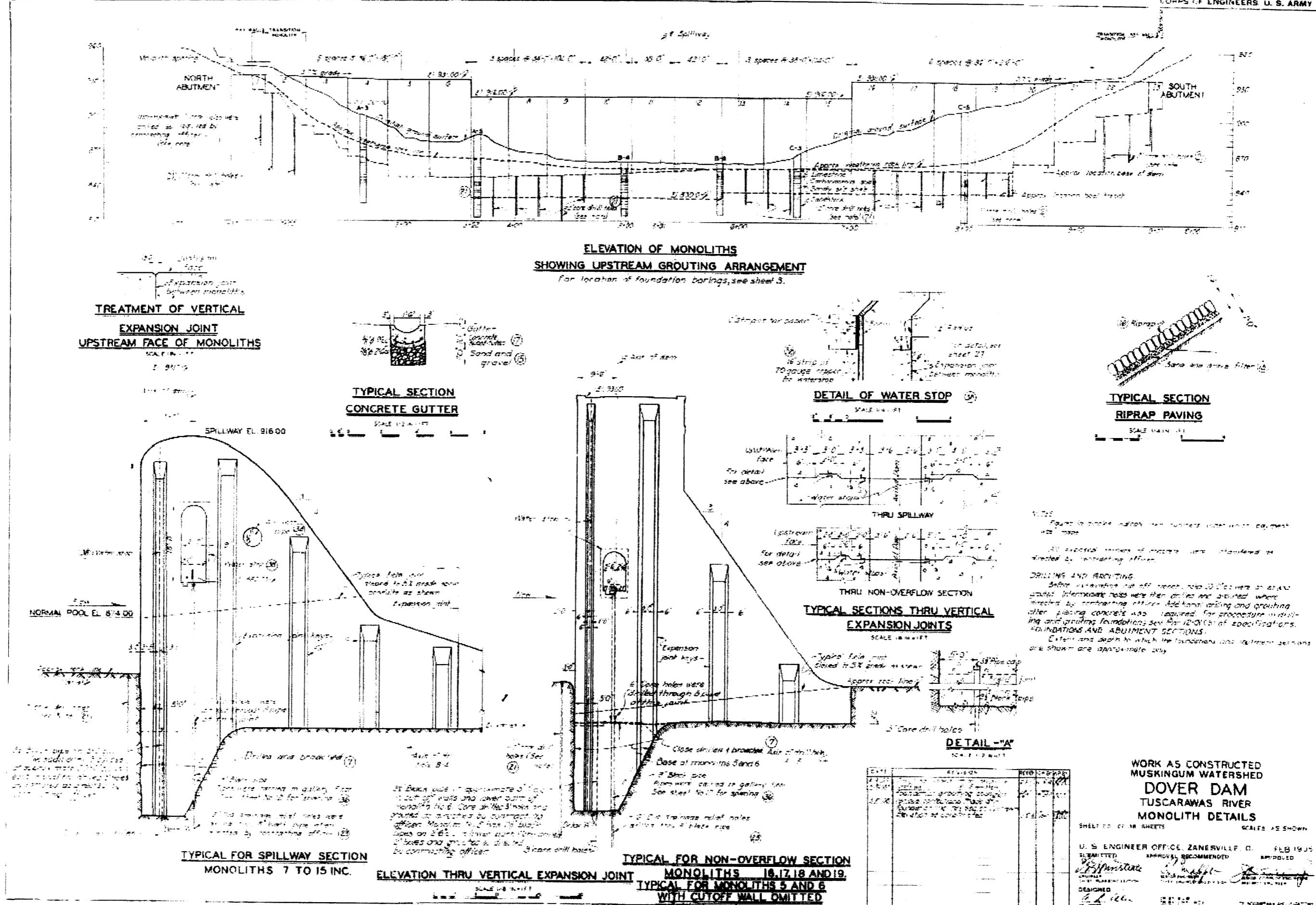
EL 934 TO EL 902

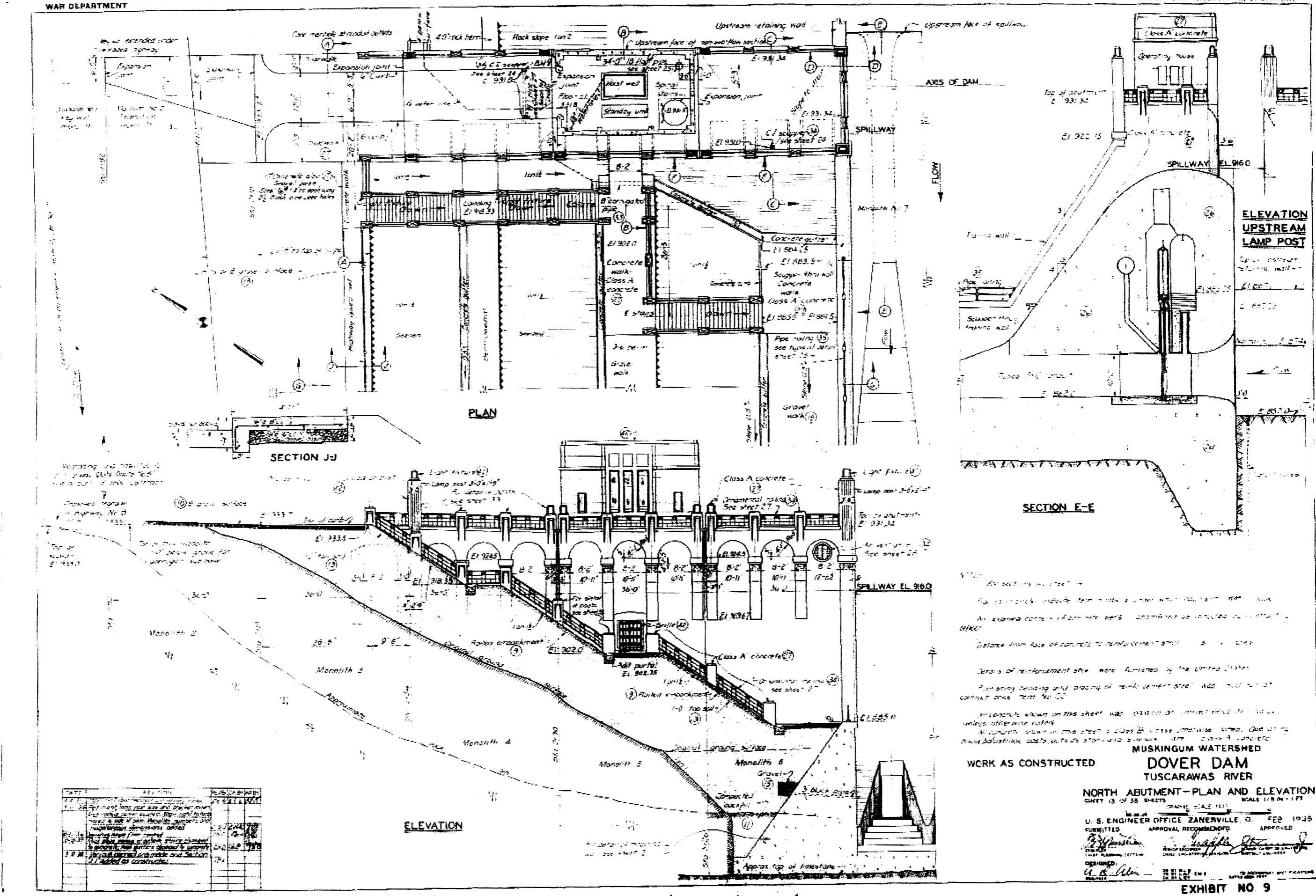
$$\text{LENGTH} = \text{STA } 936.6 + (936.6 - 921) - \text{STA } 8+08 = 150'$$

$$\text{AVG. SLOPE} = \frac{934-902}{150} = 21.3\% \text{ OR } 2.56 \frac{\text{IN}}{\text{FT}}$$

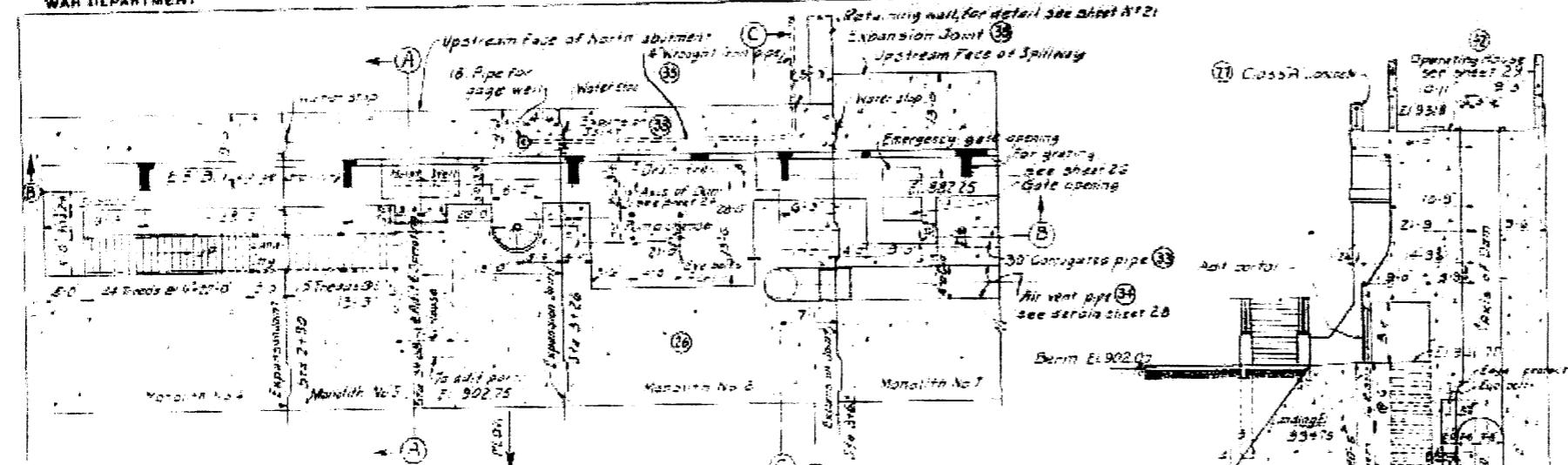
$$\text{LENGTH} = \sqrt{31^2 + 150^2} = 153.2' /$$





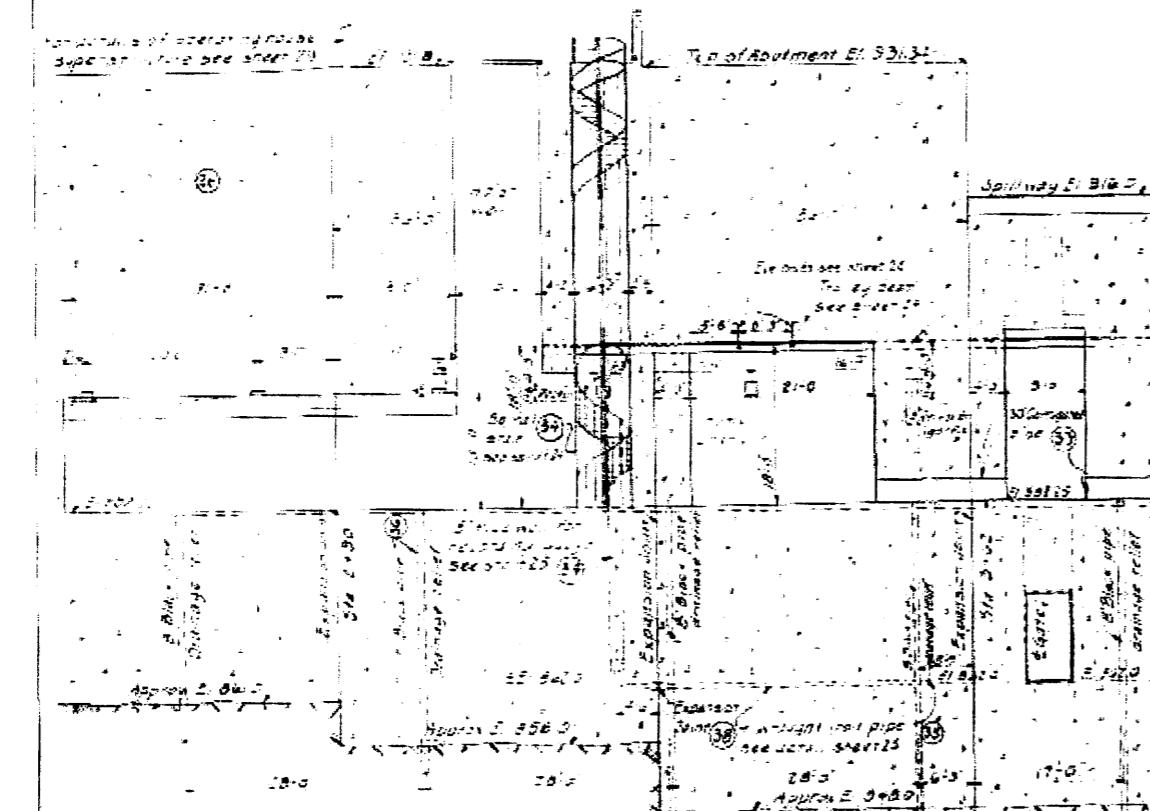


WAR DEPARTMENT



PART SECTIONAL PLAN AT EL 893.0

SCALE 1/2 IN.=1 FT



SECTION B-B

SCALE 1/2 IN.=1 FT

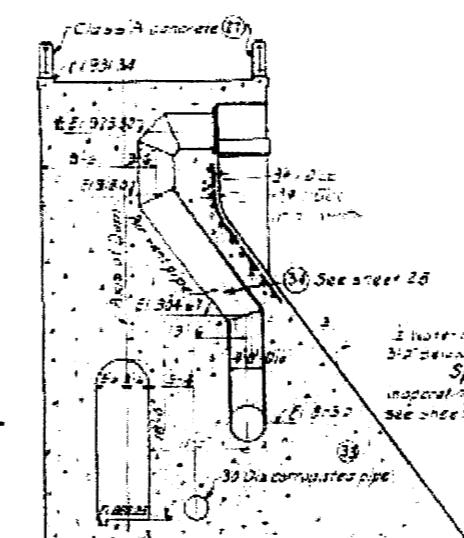
NOTES

- Figures 1000 & 1200 refer to numbers under which drawings were made.
- All exposed surfaces of concrete were reinforced as directed by Contracting Officer.
- Distance from face of concrete to top of stairs is 7 ft.
- Length of steel reinforcement bars furnished by the Contractor.
- Reinforcement steel was furnished, bent, and placed by contractor. Payment to be made under item 20.
- All concrete shown on this sheet was paid for at contract price Item 26, unless otherwise noted.

TYPICAL SECTION THRU STAIRS

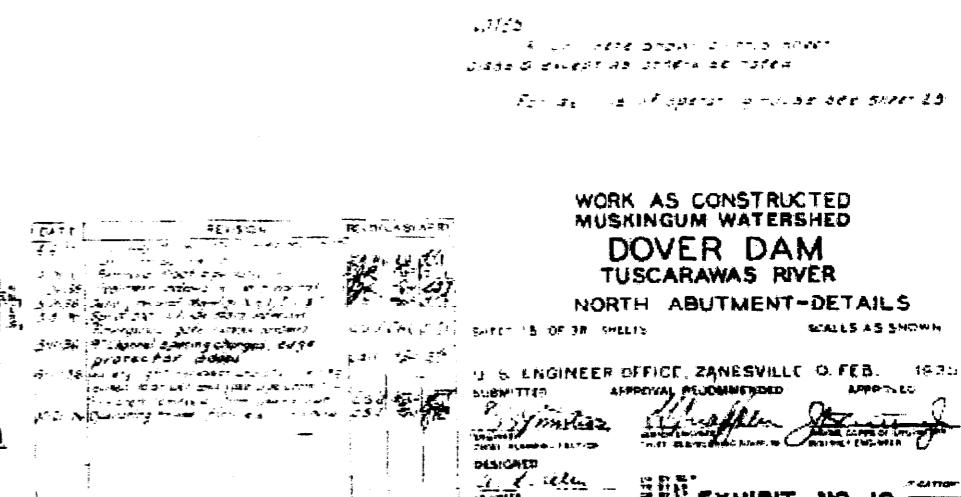
SCALE 1/2 IN.=1 FT

SECTION A-A



FLOOR PLAN OPERATING HOUSE

SCALE 1/4 IN.=1 FT

WORK AS CONSTRUCTED
MUSKINGUM WATERSHEDDOVER DAM
TUSCARAWAS RIVER

NORTH ABUTMENT-DETAILS

SCALES AS SHOWN

U. S. ENGINEER OFFICE, ZANESVILLE O. FEB. 1930

SUBMITTED APPROVED PROPOSED APPROVED

P. [Signature] J. [Signature]

DESIGNED DRAWN CHECKED APPROVED

J. [Signature] P. [Signature]

APPROVED

J. [Signature] P. [Signature]

SECTION F-F

DETAIL OF GALLERY DRAIN

SCALE 1/2 IN.=1 FT

EXHIBIT NO. 10

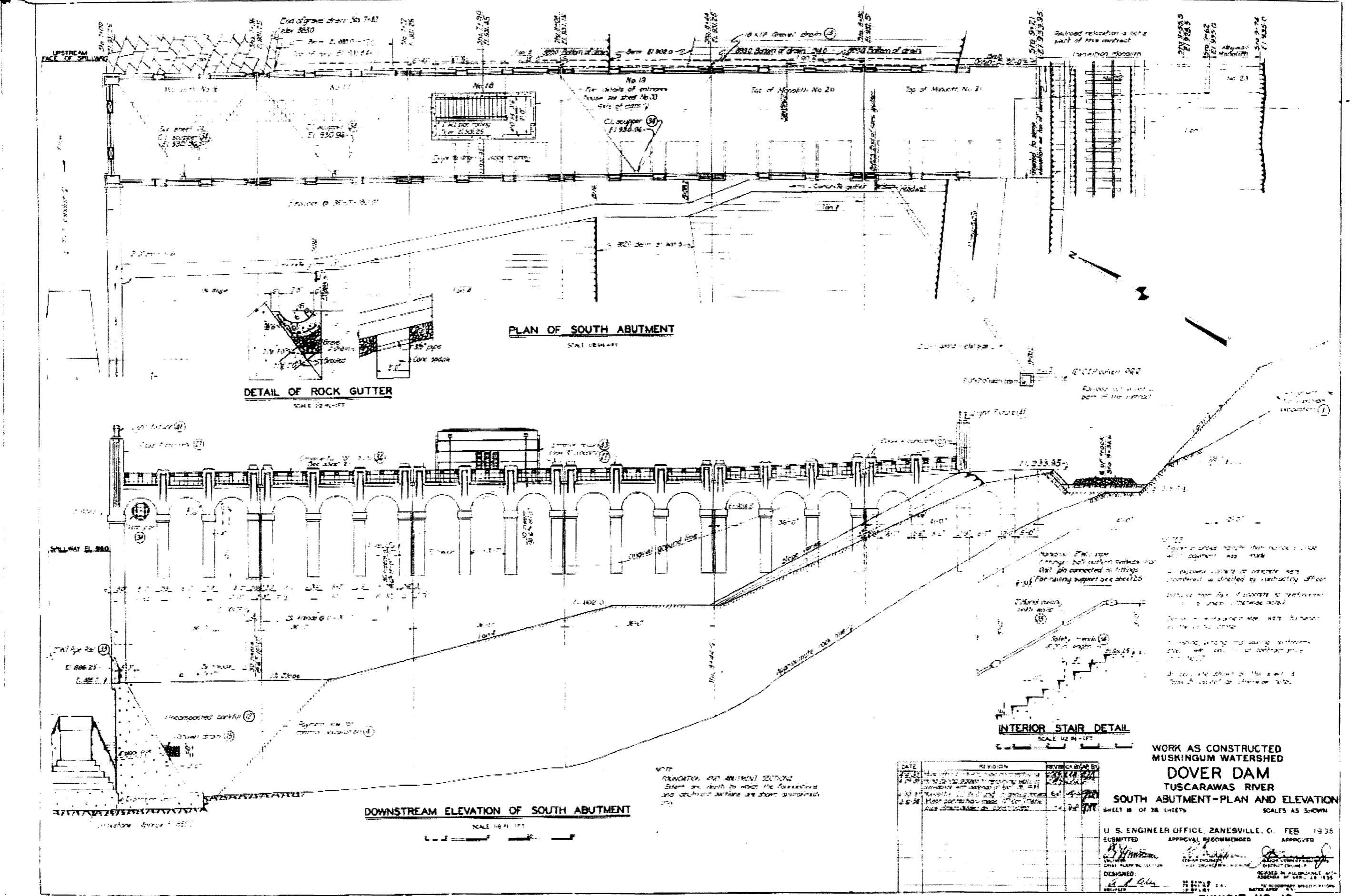
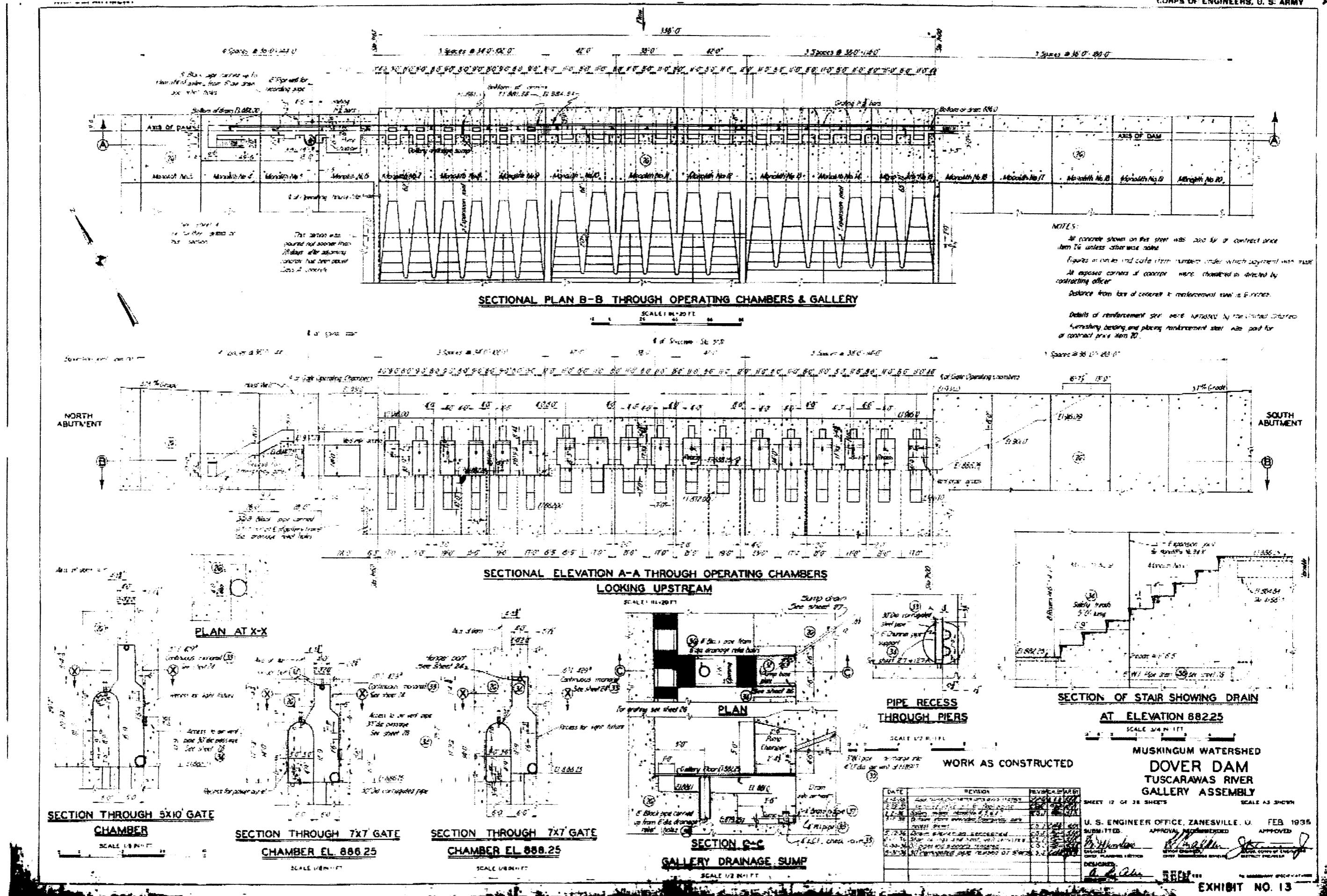


EXHIBIT NO. 12



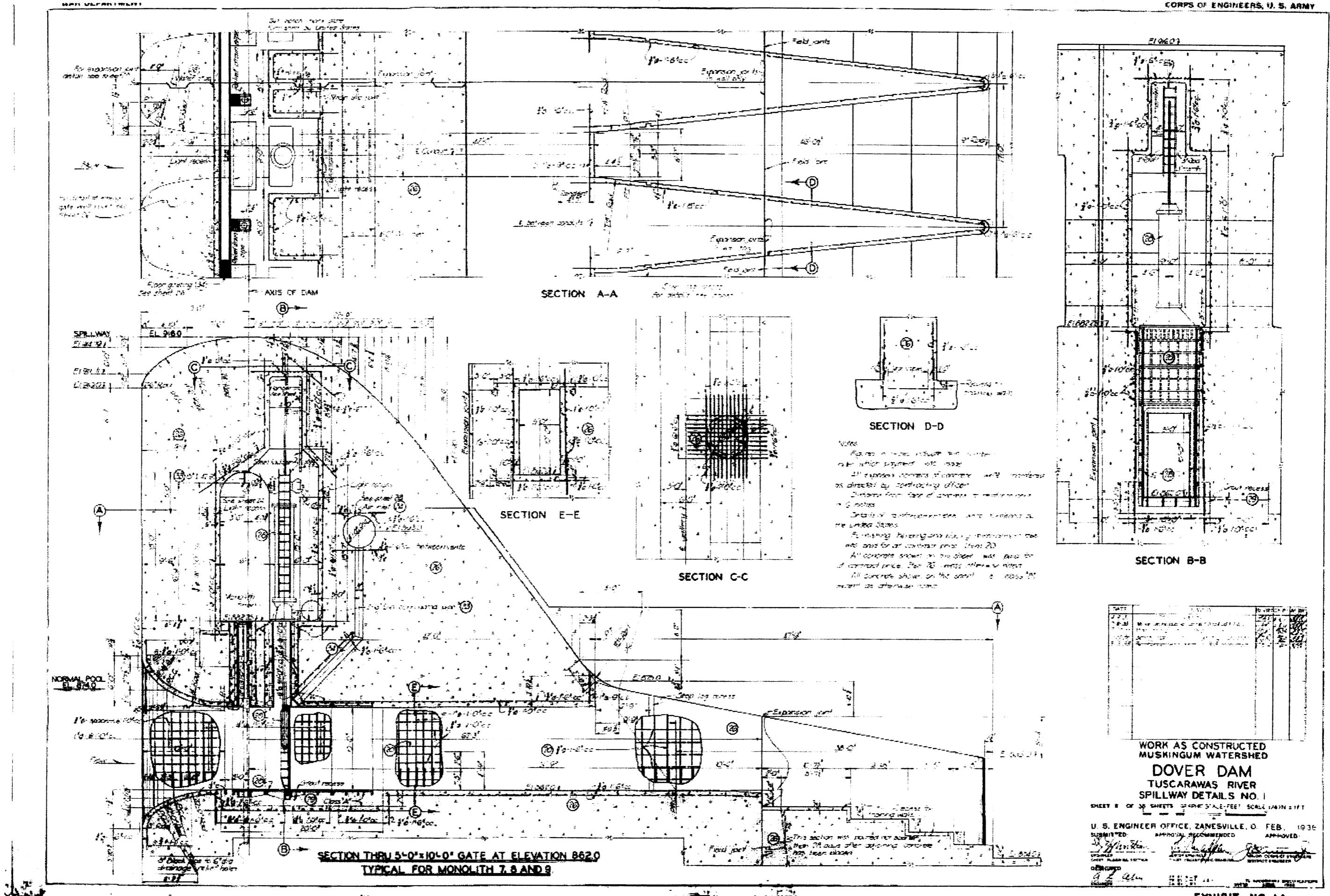
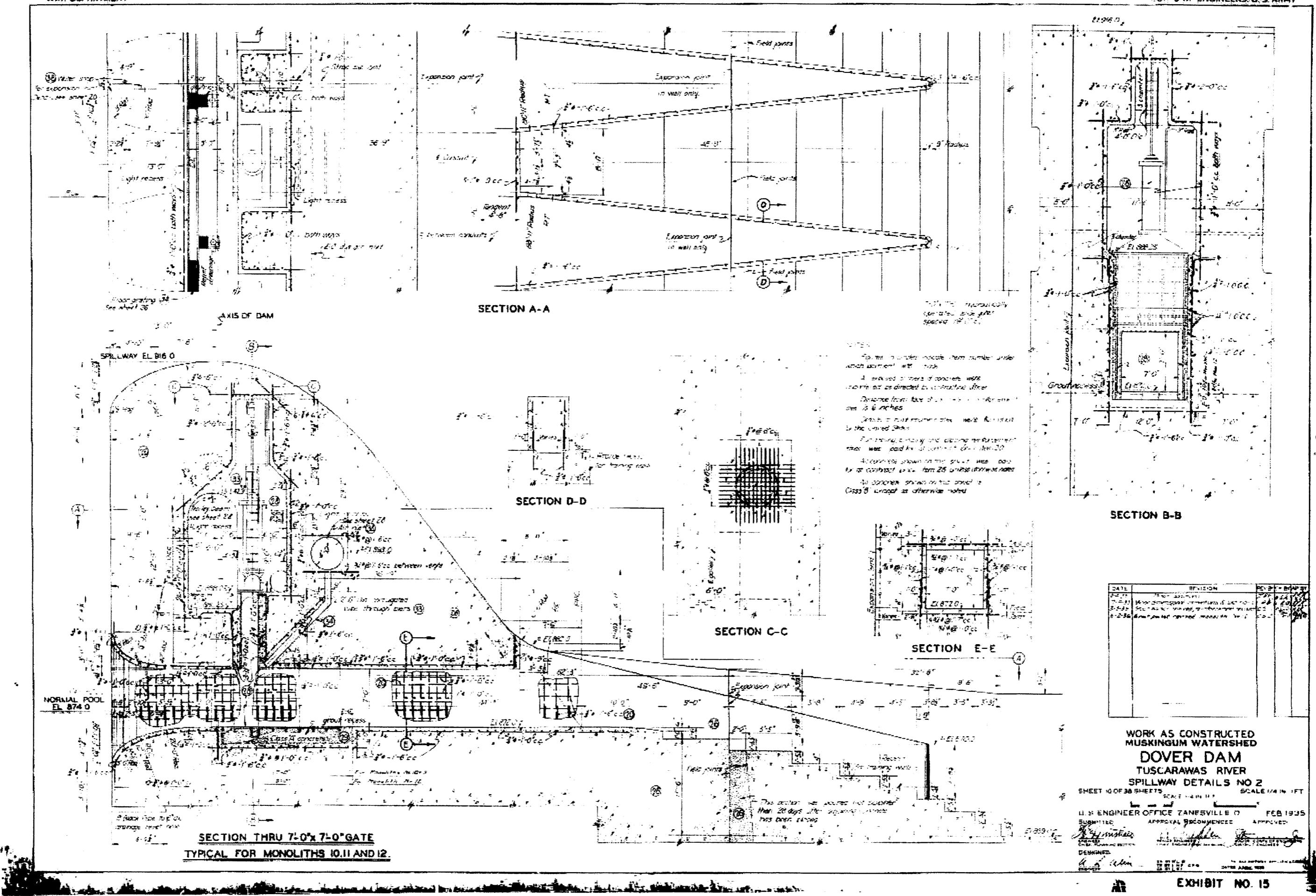
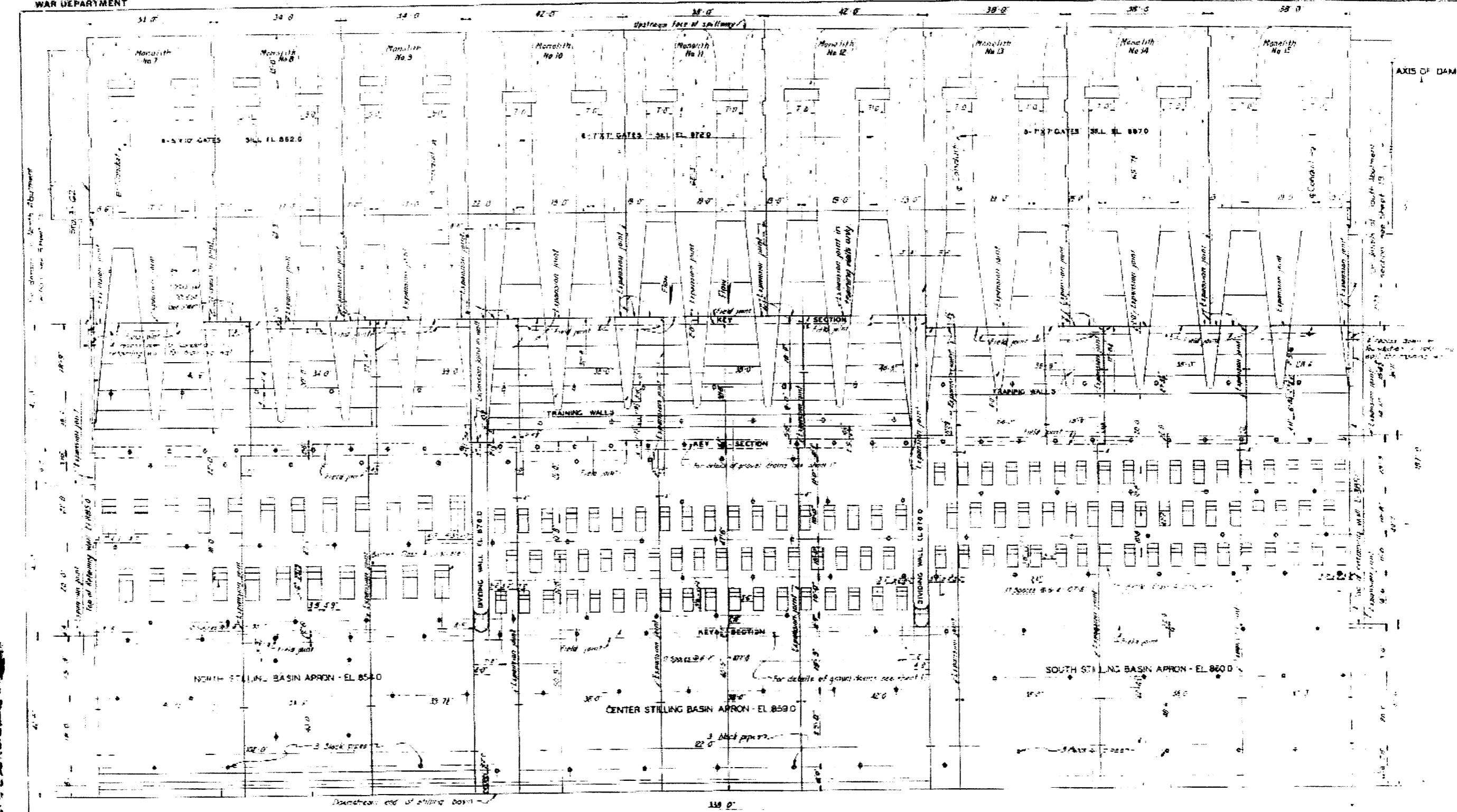


EXHIBIT NO. 14





PLAN OF STILLING BASIN

5. Stock pipe through which 3 drainage
well holes were drilled to a depth
determined by the contracting
officer.

NIST

As concrete becomes more ductile, it can better withstand lateral forces. It can also withstand greater tensile stresses under cyclic loading. As a result, the capacity of concrete structures to resist lateral forces can be increased by controlling cracks. Resistance can be increased by concrete reinforcement.

General statement was made
that the United States,
including Alaska, had
the best fire control system.
A complete statement of the
present system was given by
the Director of Fire Control, U.S.A.
He detailed the methods used
in the 32-type gun to insure
perfection.

DATE		REVISION		ED.	TYPE	PRINT	U. S. ENGINEER OFFICE ZANESVILLE O. 7-28-1931	
5-15-31		T-1		4-1	PA	PA	SUBMITTED APPROVAL RECOMMENDED APPROVED	
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**WORK AS CONSTRUCTED
MUSKINGUM WATERSHED
DOVER DAM
TUSCARAWAS RIVER**

LLING BASIN-GENERAL PLAN

30 SHEETS **100% COTTON** **WEIGHT 16 OZ.**

Figure 1. A photograph of the two sets of micrographs used in this study. The left set shows the top surface of a sample of *Leptothrix* sp. at 1000 \times magnification. The right set shows the bottom surface of the same sample at 1000 \times magnification.

ENGINEER OFFICE ZANESVILLE, O. 776-1434

INTERVAL RECOMMENDED APPROVED

10. *Chlorophytum comosum* (L.) Willd. (Asparagaceae)

EDWARD R. GOREY **ILLUSTRATOR**
PAULINE GOREY **EDITOR**

FIGURE 1 The relationship between the number of species and the area of forest cover in each of the 100 plots.

• 66 •

EXHIBIT NO. 16